FIELD TRIAL OF GRAVEL STABILIZATION METHODS

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This is the construction report for a study designed for field verification of the findings contained in FHWA HP&R Study ME-90-92 entitled "A Review and Experimentation of Gravel Stabilization Methods" published July, 1990.

Prepared for.

Maine Department of Transportation

by·

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FIELD TRIAL OF

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An Abstract of
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Gravel stabilization is a potential method of improving the performance of weak base course aggregates Previous research determined that soil-cement, asphalt, and calcium chloride stabilized subbase aggregate were potentially appropriate stabilization methods in terms of constructability, improved strength, and improved durability Thus, MDOT sponsored construction of a full-scale experimental highway section to evaluate the constructability and short term performance of these stabilization methods The ability to predict field performance from laboratory tests was also evaluated

Tests showed that loading, hauling, and grading the aggregate began the degradation process and that mixing and compaction further increased the fines content. Construction challenges inherent to each stabilization method were overcome during construction of the test section, but overcoming these difficulties in actual road construction will require planning Based on Road Rater tests, soil cement provides the largest structural benefit, with asphalt providing a lesser benefit Calcium chloride provided no discernible increase in strength compared to untreated control sections

Strength tests were performed on field generated and laboratory generated samples using aggregate from the test section The field generated soil-cement samples had significantly lower strengths than laboratory generated samples Reasons for the disparity are discussed Laboratory mixed Marshall samples produced nearly the same results as field mixed samples Therefore, laboratory mixed samples can be used to predict the behavior of field mixed aggregates CBR results from field generated calcium chloride stabilized samples were lower than laboratory mixed samples.

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CHAPTER 1

INTRODUCTION

PROBLEM STATEMENT

Sandy gravel aggregates excavated from northern Maine borrow pits and used for road construction often have poor strength characteristics The aggregate is mechanically broken down during construction processes, particularly compaction and construction traffic, before the contractor is able to pave prepared sections There are also indications that degradation may continue as a result of repeated traffic loading after construction is completed (Nunan and Humphrey, 1989)

Aggregate degradation increases the fines content of the base material immediately beneath the pavement Therefore, this layer has lower strength and is susceptible to frost action This results in premature pavement failure Pavement failure occurs primarily in the form of rutting and cracking of the pavement surface

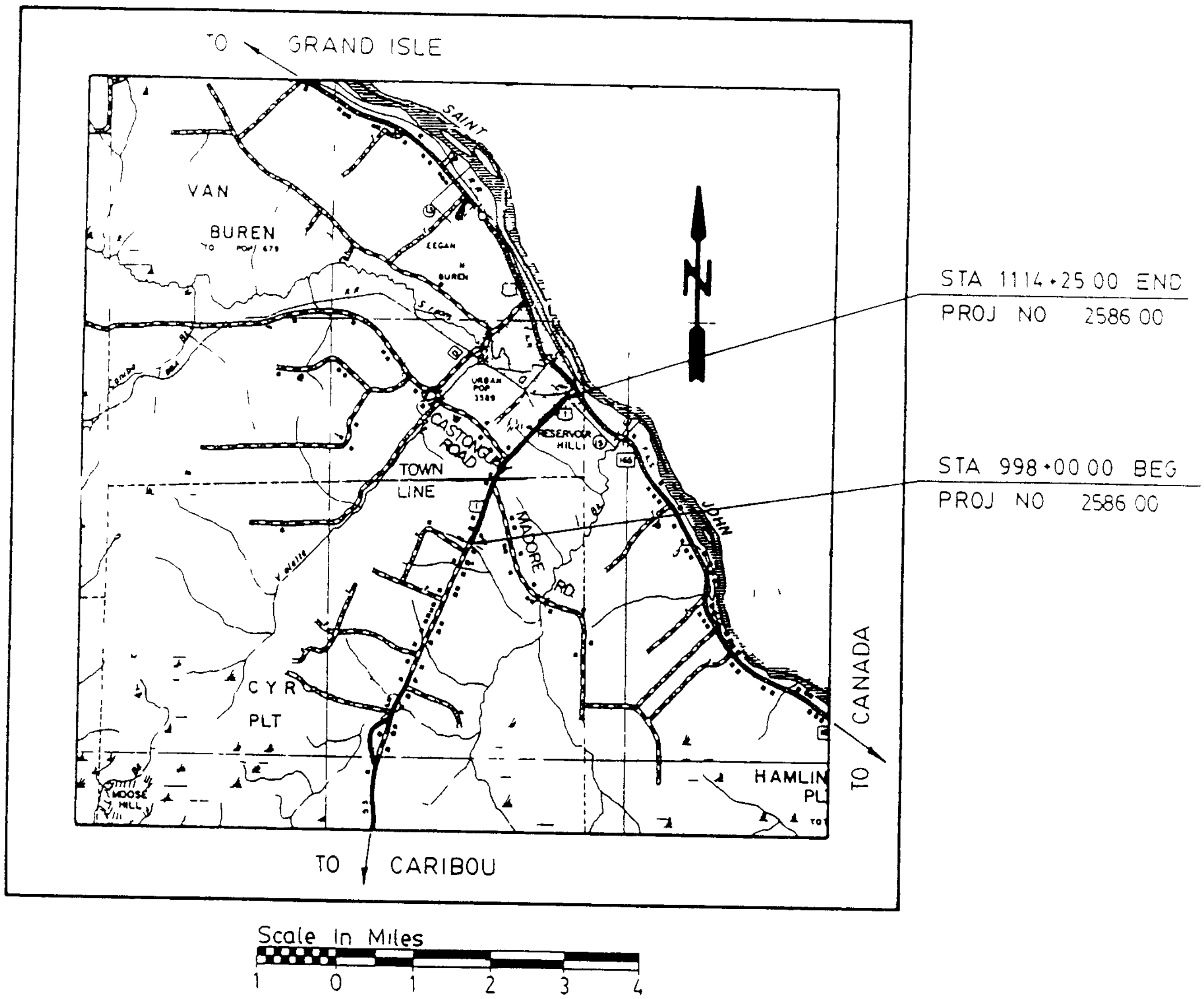
Gravel stabilization is presently being investigated by the Maine Department of Transportation (MDOT) as a potential method of improving the performance of this aggregate when used as a base material Previous research (Nunan and Humphrey, 1989) determined that several stabilization methods were potentially appropriate in terms of constructability and improved strength and durability characteristics of the aggregate Consequently, MDOT sponsored construction of a full-scale highway test section

PURPOSE OF RESEARCH

The purpose of current research is to conduct a performance evaluation of three base aggregate stabilization methods in a full-scale stabilized highway setting Secondly, this research provided an opportunity to evaluate the predictability of stabilization field performance based on comparison of test results from field-generated samples and laboratory-generated samples Finally, this project provided an opportunity to document construction of a stabilized base The information gathered will facilitate future performance monitoring of the individual stabilization methods at this site and initiates a general gravel stabilization data base

PROJECT DESCRIPTION

The experimental highway segment constructed for this full-scale field trial is located on Route 1, about two miles south of Van Buren, Maine (See Figure 1 1) The experimental highway segment is part of MDOT Project No 2586 00, a 2 2 mile long total reconstruction project The test section was constructed in mid-



A PORTION OF AROOSTOOK COUNTY

LEGEND
State Maintained
Ways and Bridges within the
Construction Area

Figure 1 1 Project Location Map

September 1990, and comprises a stabilized base and control sections with a total length of 1020 feet

The 1020 foot long experimental section began at the Project STA 1028+00 and ended at STA 1038+20 The test section consisted of 200 foot long segments of soil-cement, asphalt-stabilized, and calcium chloride-stabilized materials, as well as two control sections and one 20 foot long untreated section The stabilized and control sections were located as follows

Soil-Cement Stabilized Section	STA 1028+00 to 1030+00
Modified Subbase Control Section	STA 1030+00 to 1032+00
Asphalt Stabilized Section	STA. 1032+00 to 1034+00
Untreated Section	STA. 1034+00 to 1034+20
Calcium Chloride Stab Section	STA 1034+20 to 1036+20
Standard Subbase Control Section	STA 1036+20 to 1038+20

REPORT ORGANIZATION

The remainder of this report is composed of six chapters Chapter 2 provides background information from the limited number of stabilization projects in which performance results were documented in the literature The literature review includes a summary of the previous study conducted by Nunan and Humphrey (1989) and information gathered from a computer search through Transportation Research Board (TRB) information services

A brief discussion of construction elements common to each 200 foot segment is presented in Chapter 3 This chapter also includes a summary of construction of the two control sections

Chapters 4, 5, and 6 present an in-depth evaluation of soil-cement, asphalt, and calcium chloride stabilized base material, respectively These chapters discuss pre-construction laboratory testing, the construction process, and post-construction laboratory testing Field and laboratory results are compared and an interpretation of their significance is also presented

Research findings are summarized in Chapter 7 This chapter presents a summary discussion of the individual stabilization methods, as well as recommendations for future research and monitoring

Soil test procedures are summarized in Appendix A, Field And Laboratory Test Procedures Test results have been placed in appendices where appropriate Reference is made to "Field Trial of Gravel Stabilization Methods, Supplemental Specifications (MDOT, 1990) for the construction specifications used for the field trial

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CHAPTER 2

BACKGROUND INFORMATION

STABILIZATION METHODS

The current phase of the MDOT gravel stabilization research relied extensively on the work of Nunan and Humphrey (1989) They conducted a comprehensive review of a number stabilization methods

Nunan and Humphrey (1989) evaluated the basic mechanical and chemical principles of various stabilization methods to ascertain their applicability to stabilizing gravel base materials in northern Maine In that context, they evaluated the stabilization methods listed in Table 2.1

From this list, lime, lime-fly ash, sodium chloride, BIO CAT 300-1/EMC^2, permazyme, terrazyme, membrane encapsulated soil layers, geotextiles, and geowebs/geogrids, were eliminated because they were not applicable to the stabilization of gravel subbases or their laboratory studies showed they had limited or no beneficial effect The following text discusses the three methods that were recommended for use in this field trial soil-cement, emulsified asphalt, and calcium chloride

Table 2 1 Stabilization methods evaluated for applicability to northern Maine road construction (Nunan and Humphrey, 1989)

Soil Cement
Lime
Lime-Fly Ash
Asphalt
Calcium Chloride
Sodium Chloride
BIO CAT 300-1/EMC[2]
Permazyme
Terrazyme
Membrane Encapsulated Soil Layers
Geotextiles
Geowebs/Geogrids

Soil-Cement

Soil-cement base stabilization comprises a mixture of Portland cement, soil, and water The soil acts primarily as a filler When water is added to cement, the pozzolanic materials (silica and alumina) in the cement form a paste comprised of mono and dicalcium silicate hydrates Surface adhesion forces develop between the hydrates and the soil particles causing the soil particles to bind together (Nunan and Humphrey, 1989) Thus, when compacted and cured, the stabilized material has increased strength and durability qualities

Cement stabilization works with all types of soil However, sands and sandy gravels are particularly responsive to cement stabilization All cement types may be used for stabilization Type I cement is typically used because Types II, III, and V produce nearly the same effect on most soils and because Type I cement is readily available and economical (Nunan and Humphrey, 1989) Thus, Type I cement was chosen for this field trial

Unconfined compression, freeze-thaw, and wet-dry tests are typically used to measure the strength and durability of soil-cement mixtures Nunan and Humphrey (1989) evaluated soil-cement stabilization of aggregates from several northern Maine regions with these tests The proven strength gain in sandy gravels and availability of Type I cement made soil-cement a logical choice for a full-scale field trial in northern Maine

Asphalt

The three forms of asphalt which have been used for soil stabilization are asphalt cement, cutback asphalt, and emulsified asphalt Due to high viscosity, these materials require heat or dilution prior to mixing with soil

For adequate mixture, asphalt cement requires high temperatures in a batch plant to sufficiently reduce its viscosity and to heat the aggregate Cutback asphalts are expensive to use because of the high cost and limited availability of the solvents used Environmental concerns arising from the release of volatile organic compounds while the mixture cures is another disadvantage For these reasons, Nunan and Humphrey (1989) made no further considerations of these types of asphalt They then focused on the use of emulsified asphalt

Emulsified asphalt is made up of asphalt cement, water, and an emulsifying agent which disperses and suspends the asphalt in the form of droplets until they come into contact with the aggregate Emulsified asphalt will stabilize most soil types, but is particularly effective in stabilization of sands and sandy gravels Soil stabilized with emulsified asphalt is usually mixed-in-place

No chemical reaction is involved with asphalt stabilization The asphalt both waterproofs and binds the soil particles together The increased shear strength of the stabilized mixture is mainly the result of increased cohesion The strength of the bond between the soil particles and the emulsified asphalt depends on whether the emulsion is cationic or anionic. The electrochemical bonds between the cationic emulsion and aggregate are stronger than the physical bonds between anionic emulsion and aggregate (Nunan and Humphrey, 1989)

The modified Marshall or Hveem method are typically used to evaluate the stability of asphalt stabilized soil Modified Marshall stability values will generally increase, reaching a peak, and then decrease with increasing asphalt Modified Marshall stability will typically decrease with increasing water content The distribution of the bitumen is less uniform with increasing moisture content (Nunan and Humphrey, 1989)

Nunan and Humphrey (1989) performed modified Marshall tests on asphalt stabilized aggregate from several northern Maine regions They stabilized the aggregate with MS-4, a medium setting anionic emulsified asphalt They easily achieved greater than the minimum stability of 500 lb recommended by the Asphalt Institute (1974) for medium traffic roads Based on their tests, availability of emulsified asphalt, and a comparison of results reported by others, Nunan and Humphrey (1989) recommended emulsified asphalt for a field stabilization trial

Calcium Chloride

Calcium chloride is a white, odorless, non-staining salt supplied in either liquid or flake form Calcium chloride is generally used to stabilize granular materials Calcium chloride stabilizes aggregate by controlling moisture Calcium chloride is deliquescent which means that it dissolves in the moisture from the aggregate and becomes a liquid highly resistant to evaporation This maintains the moisture content during compaction and creates a solution with a higher surface tension than water This, combined with the lubrication properties of the calcium chloride solution, produces higher maximum dry densities than in untreated soils given the same compactive effort (Nunan and Humphrey, 1989)

A second benefit of calcium chloride stabilization is the cation exchange between the calcium ions and the clay mineral cations present in the fines This causes a reduction in the thickness of the adsorbed water layer surrounding the clay minerals This condition helps prevent free water entry into the aggregate layer that would otherwise reduce the resistance of the aggregate to freeze-thaw and wet-dry action (Nunan and Humphrey, 1989)

Nunan and Humphrey (1989) performed laboratory tests on calcium chloride stabilized aggregate from several regions in northern Maine Treated soil strength measured by CBR tests showed varying but higher strengths over untreated soils The ease of application and potential for soil improvement indicated that this stabilization method should be considered for a field trial

PERFORMANCE OF STABILIZATION METHODS

Review of New England and Canada

Nunan and Humphrey (1989) conducted a survey of state transportation departments in the northeast and selected Canadian transportation agencies to determine their experience with various stabilization methods The remainder of this section summarizes the survey data they collected regarding performance of soil-cement, asphalt, and calcium chloride stabilized base materials Note that New Hampshire and Vermont DOT's did not respond to their survey

Connecticut has done very little gravel stabilization They used calcium chloride stabilization about six years ago and more recently tried calcium chloride and geosynthetics They found no significant difference between treated and untreated roads and have therefore abandoned the use of chloride stabilization

Massachusetts has large supplies of durable gravel and consequently has little need to investigate stabilization methods However, they have used soil-cement stabilization on several projects They found that lack of experience with soil-cement construction procedures resulted in difficulty in completing mixing and compaction within the recommended two to four hour time frame Because of these difficulties, MASSDPW will not consider soil-cement stabilization for future projects

New York tried soil-cement on an 18 mile long county road reconstruction project At this project, NYDOT incorporated 8 to 10 percent cement into the base materials and subsequently experienced significant cracking NYDOT has also used emulsified asphalt in the Lake George area They applied 5 to 6 pounds of emulsified asphalt per cubic foot of base gravel and placed the mixture in two 9-inch lifts After twenty years of service, this road is still in good condition, but NYDOT feels that untreated base gravel would have performed as well.

Rhode Island has not used any stabilization method for their base materials

The Canadian province of Alberta has used several stabilization methods with mixed success They have used cement to stabilize sands in areas where aggregate is scarce Soil-cement has performed satisfactorily except for cracking of the pavement surface. Asphalt stabilization is part of their regular staged construction practice. The Alberta DOT places a 2-inch thick layer of asphalt stabilized soil over the base course which is used as the initial wearing course. The initial wearing course is pressed into service for one to four years before the road is paved with hot-mix asphalt.

New Brunswick, Canada has used soil-cement and geosynthetics to stabilize base materials for several years. They apparently are achieving satisfactory results since the New Brunswick DOT continues to use the method. The geotextile is used primarily as a separation layer placed at the subgrade-base contact. New Brunswick also tried asphalt stabilization but has not used the method for over five years.

The Quebec Ministry of Transportation used soil-cement and asphalt stabilized materials over 15 years ago, but found the methods expensive. They also had problems with inadequate equipment, a heterogeneous final mixture, and weather constraints which limited construction time.

Nunan and Humphrey (1989) investigated the performance of a number of stabilization projects in Maine. The Maine DOT has used soil-cement, asphalt, and calcium chloride base stabilization.

Where soil-cement stabilization was used, MDOT specified a

6-inch thick stabilized base layer immediately beneath the pavement The cement content typically specified for these projects ranged between 7 and 9 percent by weight of soil Nunan and Humphrey (1989) found that the actual treated depth was less than 6 inches in every case They also found that the average cement contents of treated base materials they sampled were often less than specified Despite these construction shortfalls, the soil-cement treated sections generally performed as well or better than the untreated sections This conclusion is based on visual inspection of rutting and cracking distress, a review of maintenance records, and Road Rater deflection measurements

There was extremely limited data regarding the asphalt and calcium chloride stabilized base projects The most that can be said about these projects is that the treated sections are performing satisfactorily, but there is too little information for comparison

Computer Search of Transportation Research Board Records

The authors conducted a computerized search of TRB files through the Maine DOT Other examples of field trials where different methods of stabilization were compared would be useful in evaluating the Van Buren field trial Unfortunately, the authors found no reports of such comparative studies

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CHAPTER 3

COMMON CONSTRUCTION ELEMENTS AND CONTROL SECTIONS

SITE DESCRIPTION

Land along the 1000 foot experimental section is largely cultivated agricultural fields There are two residences and one potato barn on the west side of the test section, approximately between STA 1029+00 and 1036+00 The roadway test section follows the eastern edge of a ridge with a nearly level grade However, topography along the section grades moderately from west to east

MDOT conducted several test pits and borings in the vicinity of the experimental highway The subsurface investigations indicated 3 to 7 inches of asphalt underlain by 8 to 18 inches of sandy gravel with some silt and occasional cobbles Beneath the gravel layer, MDOT found 1 to 4 feet of glacial till consisting of sandy, gravelly, silt Weathered phyllite bedrock was found below the till None of the borings or test pits encountered groundwater However, the groundwater level will fluctuate with seasonal variations, freeze-thaw, adjacent construction activity, and run-off

COMMON CONSTRUCTION ELEMENTS

Subbase Preparation

Subgrade and subbase preparation were identical for all the stabilized and control sections The contractor excavated the existing roadway down to the desired subgrade elevation and compacted the exposed subgrade with a vibratory roller Then 19 to 20 inches of subbase aggregate meeting MDOT Specification 703 06b, Type D, was placed in two lifts of approximately equal thickness This aggregate has a maximum allowable particle size of 6 inches Each lift was compacted with a vibratory roller to a minimum of 95% of the maximum dry density determined in accordance with AASHTO T 180 (ASTM D 1557) These construction operations conformed with MDOT Specification 304

Vehicular and construction traffic traveled on the exposed surface for several weeks prior to construction of the test sections Just prior to construction of the test sections, traffic was diverted to the shoulders beyond the limits of the stabilized and control sections The upper few inches of the prepared subbase had been degraded by the traffic Therefore, the upper 1 to 2 inches of subbase aggregate was graded to the shoulders with a Komatsu GD 605 A road grader, leaving an 18-inch thick subbase within the limits of the test sections There may have been some limited areas with partially degraded aggregate remaining The exposed surface was then recompacted with an Ingersoll Rand Model SP 56 DD vibratory roller

Samples of the standard subbase aggregate from each test section were collected before the contractor placed the modified subbase aggregate Samples of standard subbase aggregate were also obtained from the completed standard subbase aggregate control section Laboratory grain size distribution tests were performed on the standard subbase aggregate samples Test results are presented in Appendix B, Figures 8 through 12 The fines content ranged between 5 6% and 9 5% This aggregate has been subjected to both compaction and traffic loading It is not possible to attribute what amount of degradation is the result of either types of loading It is apparent, however, that degradation of the aggregate has begun and suggests potential frost susceptibility

Modified Subbase Aggregate Placement

The stabilized base course was constructed with modified subbase aggregate This aggregate was similar to standard subbase aggregate (MDOT Specification 703 06b, Type D, 6-inch maximum size) except that a 2-inch maximum size was used This smaller maximum size was necessary to permit subsequent mixing operations The specified gradation of the modified subbase aggregate is shown in Table 3 1 The actual grain size distribution of the modified subbase aggregate determined from samples taken from the project stockpile is presented in Appendix B, Figure 1 This material had about 4 5% passing the No 200 sieve

The stabilized base course was constructed between 10 and 12 September 1990 During that period, three stabilized sections and the modified subbase control section were constructed The contractor had previously completed the standard subbase control section The contractor began by loading the modified subbase aggregate from the stockpile with a Caterpillar 966 front end loader at what was called the "LaPlante Pit" in Cyr Plantation and hauled it to the site in 16 and 20 yard wheeler dumps The borrow area was about 1 mile from the field trial site

Table 3 1 Modified subbase aggregate grain size specification

Sieve Designation	Percentage by Weight Passing Square Mesh Sieves
2-inch	100
1/4-inch	25-70
No. 40	0-30
No 200	0-7

When the modified subbase aggregate reached the site, the contractor dumped it on the prepared standard subbase course, spread it with a Caterpillar D-3 or D-4 dozer, and lightly compacted it with a static smooth drum roller This course was graded and rerolled as needed to produce a 6 5 to 7 inch thickness of lightly compacted material This was judged to be sufficient to produce a 6-inch thick layer compacted to the specified density

Samples of the modified subbase aggregate were collected from each of the stabilized sections and from the modified subbase control section after placement and grading but before compaction

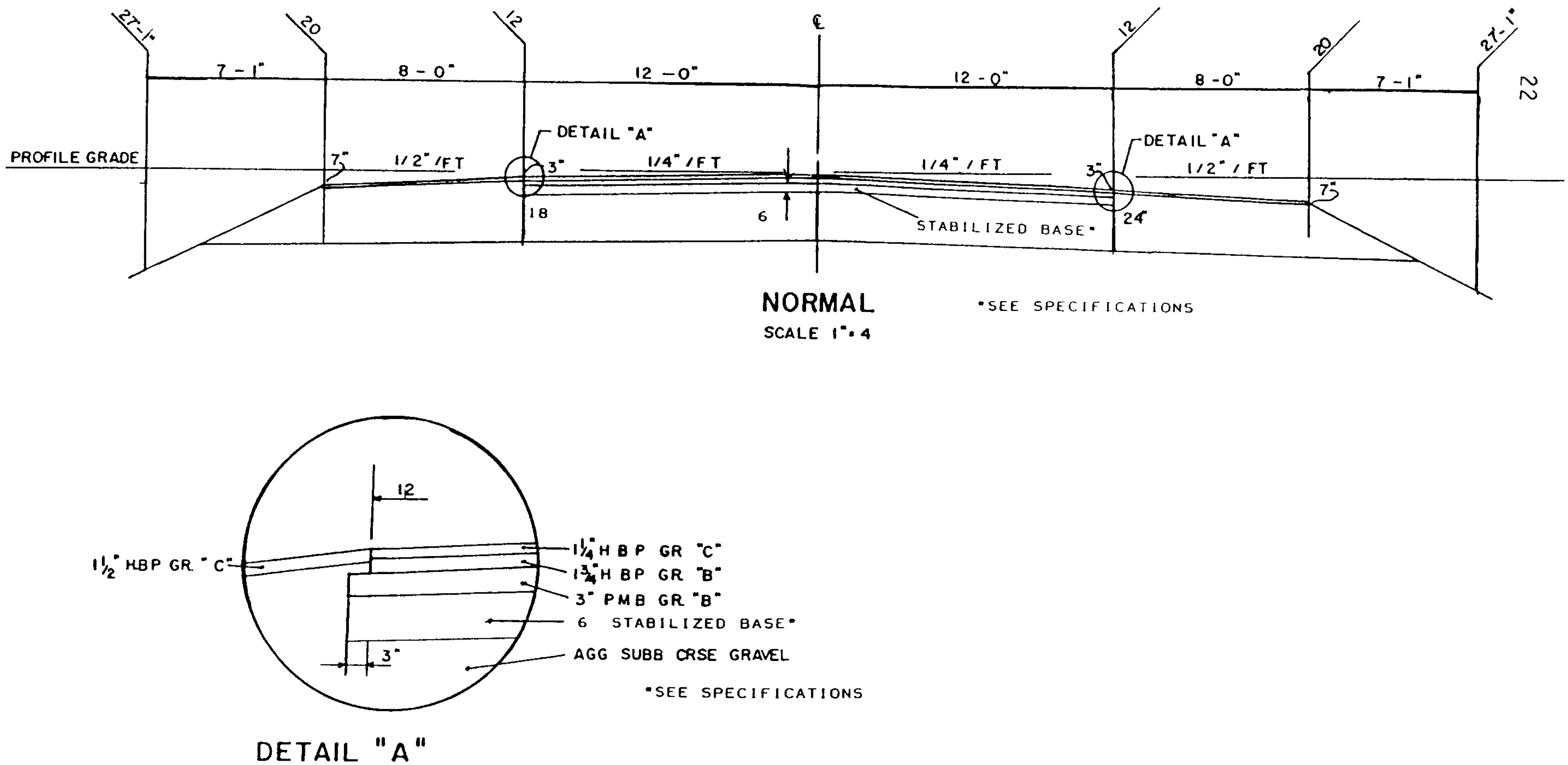
Results of grain size distribution tests of these samples are presented in Appendix B, Figures 2 and 5 The fines content of this aggregate ranged between 4 7% and 6 4% This is an increase in the fines content as compared to the 4 5% fines found in the same aggregate sampled from the stockpile This is another indication that loading, hauling, placing, and grading of this particular modified subbase aggregate begins the degradation process

The next step in the construction sequence was different for each stabilization method and the two control sections A discussion of the construction of the control sections is presented in the next section A separate chapter is devoted to each of the stabilization methods

Paving Operations

When construction of the stabilized and control section aggregate base course was complete, the contractor placed a 3-inch thick layer of hot-mix, base course, asphalt concrete This layer was followed by a second 1-3/4-inch thick binder course layer Traffic used this "binder" pavement until late spring, 1991, when the contractor placed the final asphalt wearing course layer The completed typical cross-section is as shown in Figure 3 1

The contractor used a standard paving machine to place the two asphalt concrete base course layers In combination, the contractor used a Buffalo Springfield 13-1/2 ton finish roller, a Dynapac 12 ton rubber tire roller, and a Hyster C 350 C 14 ton dual drum roller



TYPICAL SECTION

Fig 3 1 Completed cross-section profile

to compact the asphalt Liquid asphalt bleeding through the 3-inch base course layer was observed in a strip about 3 feet wide in the center of the northbound lane from STA 1032+30 to 1032+85 and 1033+25 to 1033+40 This bleeding may be attributed to the pavement and not the test section

The 1-1/4-inch thick asphalt concrete wearing surface was placed in 1991 after the completion of this research project

Road Rater Pavement Deflection Measurements

The Road Rater is a van-mounted, non-destructive testing vehicle equipped with four sensors spaced one foot on center Through a combination of static and dynamic loading, the sensors record the vibrational response of the various layers of the pavement structure. Sensor 1 measures deflections at the surface of the pavement layer and provides a measure of the overall strength of the road Sensors 2, 3, and 4 measure the deflection at one foot intervals below the surface of the pavement (Nunan and Humphrey, 1989) The test parameters of this vehicle are 1 5 kips and 25 Hertz (MDOT)

Pavement deflections were measured on 28 September 1990, 21 May 1991, and 6 August 1991 The measurements were made at the surface of the pavement in the outer wheel path The deflections were recorded in mils (0 001 inch) Although no deflection criteria exist for interpreting the results, a deflection exceeding 5 mils is considered undesirable (Nunan and Humphrey, 1989) Computer printouts of the field data and computer plots of the reduced field data are presented in Appendix E, Road Rater Deflection Measurements The average road rater deflection measurements are presented in Table 3 2 All of the average sensor 1 (pavement) measurements indicated less than 5 mils deflection All of the subgrade measurements (sensors 2 to 4) indicated about 3 mils or less deflection The response curves for the soil cement stabilized section indicated the smallest deflection for each measurement event The response curves for the other sections appear to be relatively similar

It was judged that frost was out of the ground for the May 1991 Road Rater event The May 1991 measurements indicated average sensor 1 deflections of about 3 8 mils or less for both the stabilized and control sections In this Road Rater deflection measurement event, the soil-cement and asphalt stabilized sections appeared to have the smallest deflection responses, evidence of improved stability The August 1991 road rater event produced the smallest deflection measurements of all three events Note that the total pavement depth was 3, 4-3/4, and 6 inches for the 28 September, 21 May, and 6 August Road Rater events, respectively

Post-Construction Laboratory Testing

In general, there were three goals to accomplish through post-construction laboratory testing The first goal was to measure strength and durability characteristics of samples prepared from mixed in-place stabilized aggregate This data would allow evaluation of the initial performance of each stabilization method

Table 3 2 Average road rater deflection measurements (mils)*

Date	Section	Sensor 1	2	3	4
9-28-90	Soil Cement	2 30	1 87	1 24	0 64
5-21-91		2 52	2 03	1 23	0 76
8-6-91		1 37	1 04	0 75	0 42
9-28-90	Modified Sub	4 15	2 78	0 98	0 45
5-21-91		3 83	2 70	1 19	0 60
8-6-91		2 01	1 31	0 72	0 37
9-28-90	Asphalt Stab	4 18	2 69	1 08	0 46
5-21-91		2 79	2 00	1 01	0 49
8-6-91		1 60	1 12	0 70	0 36
9-28-90	CACL-2 Stab.	4 21	2 99	1 27	0 73
5-21-91		3 18	2 36	1 29	0 74
8-6-91		2 05	1.44	0 93	0 55
9-28-90	Standard Sub.	4 49	3 06	1 21	0 42
5-21-91		3 55	2 34	1.11	0 53
8-6-91		2 04	1 31	0 73	0 36

* Total Pavement Depth For Each Road Rater Event

9-28-90	3 Inches
5-21-91	4-3/4 Inches
8-6-91	6 Inches

The second goal was to make laboratory mixed samples with precisely the same constituents and moisture content as the field generated samples These samples would be used in comparative strength and durability tests Comparing results from field and laboratory mixed samples would provide insight as to whether or not strength and durability qualities of stabilized aggregate could be predicted by laboratory testing

The third goal was to track aggregate degradation by performing grain size distribution tests at various stages of construction This data will initially be used to determine what processes and to what extent the aggregate is degraded during construction In the future, this data will provide the backdrop for evaluating the effects of traffic loads on the aggregate

The results of these tests are discussed in the chapters on each stabilization method

CONTROL SECTIONS

Modified Subbase Control Section

Two control sections were constructed These sections will facilitate comparative evaluations of post-construction pavement performance and aggregate degradation between stabilized and non-stabilized soil materials The first control section consisted of a 6-inch thick course of modified subbase aggregate placed on top of the prepared subbase course This control section was located between STA 1030+00 and 1032+00

The minimum compaction was specified at 95% of AASHTO T 180 (ASTM D 1557) The actual percent compaction ranged between 96 5% and 99 4% and the placement water content ranged between 7 9% and 8 4% as determined with a nuclear density gauge The percent compaction values were based on an estimated maximum dry density of 135 0 pcf The maximum dry density was determined to be 134 9 pcf in a laboratory moisture density Thus, the actual percent compaction is slightly higher than above Grain size distribution tests of modified subbase aggregate samples from this control section are discussed in the Common Construction Elements section

The modified subbase control section was inspected just before the contractor began paving on 19 September 1990 Traffic was allowed to travel over the compacted course for about 36 hours prior to placing the binder Slight raveling of the aggregate surface was noted in the northbound lane wheel paths with a maximum depth of about 1/2-inch The paving operations and Road Rater pavement deflection measurements are discussed elsewhere in this chapter

In addition, an untreated zone was included from STA 1034+00 to STA 1034+20 to separate two adjacent stabilized sections The untreated zone was constructed in a manner identical to the modified subbase aggregate control section

Standard Subbase Control Section

The second control section was constructed from STA. 1036+20 to 1038+20 using only standard subbase aggregate, i.e., MDOT Specification 703.06b, Type D, with 6-inch maximum size. The aggregate was spread on the prepared subgrade in two lifts of approximately 13 inches each. Each lift was compacted with a vibratory roller. Traffic traveled on this section for over a month prior to paving, but the upper few inches of degraded aggregate was graded to the shoulders just prior to paving. This left 24 inches of compacted standard subbase. Thus, this control section was identical to the subbase used for the road beyond the limits of the test area.

Minimum compaction for the standard subbase control section was specified at 95% of AASHTO T 180 (ASTM D 1557). The actual percent compaction ranged between 93.0% and 101.0% and the water content at the time of testing ranged between 5.0% and 5.8% as determined with a nuclear density gauge. The percent compaction values are based on a maximum dry density of 135 pcf as determined by MDOT. Of the four tests performed, only one failed to meet the specified compaction level. That particular test, however, was run twice due to erroneous initial results. It is possible that large (6-inch) aggregate may have produced spurious readings by the nuclear gauge at this location. Grain size distribution tests of standard subbase aggregate samples from this control section are discussed in the Common Construction Elements section above.

The condition of the standard subbase control section was observed just before the contractor began his paving operation Rutting of the aggregate surface was noted between STA 1036+00 and 1036+40 in this control section This is the area where the calcium chloride distribution truck stopped after applying calcium chloride to the previous experimental section The tank trailer was designed with a shut-off valve at the front end of the trailer This allowed the 3-inch distribution line along the full length of the tank belly to drain after closing the valve

As a result, a significant amount of calcium chloride was released between the above-referenced stations The aggregate in this area was wet and soft and was easily rutted Nevertheless, the resident engineer allowed pavement placement over this area because the binder course would be used as the wearing surface through the winter If necessary, this section would be shimmed in the spring before the final wearing course was placed

No significant raveling was observed in the standard subbase control section However, calcium chloride discoloration of the aggregate surface was noticeable over 100% of the northbound lane and 75% of the southbound lane There did not appear to be any surface deflection when loaded wheeler dump trucks traveled over the section The paving operations and Road Rater pavement deflection measurements are discussed elsewhere in this chapter

SURVEYED CROSS-SECTIONS

MDOT surveyors performed cross-section surveys at 100 foot intervals over the length of the test area to document the as-built cross-section and for use in future performance evaluations The cross-sections were made at the following stations 1028+50, 1029+50, 1030+50, 1031+50, 1032+50, 1033+50, 1034+50, 1035+50, 1036+50, and 1037+50

At each of these stations, an elevation profile was made at six levels in the pavement structure as construction progressed Each profile was made with shots taken at one foot intervals from the centerline except for the profile at the top of the binder immediately before placement of the wearing course where shots were taken at approximately 11 foot intervals. Profiles were made at the following pavement structure levels 1) top of prepared subgrade, 2) 12 inches above the top of prepared subgrade in the standard subbase aggregate course, 3) top of stabilized or control course just prior to placing the binder, 4) top of binder immediately after paving, 5) top of binder immediately before placement of the wearing course, and, 6) top of completed wearing course

The profile that was 12 inches above the top of the prepared subgrade was within the subbase course It was necessary to delineate the profile so that it could be located at a future date This was done placing a 1-inch thick layer of 1/2-inch crushed stone placed on top of the surveyed profile The stone layer is about five feet wide across the full 24-foot traveled way The surveyed cross-sections are presented in Appendix C

The surveyed cross-sections will permit the future evaluation of the magnitude and source of rutting If significant rutting is observed at the surface, it is anticipated that excavations will be made to determine how deep the rutting propagated through the road structure Knowing the source of the rutting will assist in formulating appropriate modifications to the design

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CHAPTER 4

SOIL-CEMENT STABILIZED BASE

PRECONSTRUCTION LABORATORY TESTING

The amount of cement needed to stabilize the base soils and compaction control data had to be determined prior to construction of the experimental section Therefore, compressive strength tests on soil-cement stabilized base materials were conducted using 6, 7, and 8 percent cement by weight The tests were performed on modified subbase aggregate mixed with Type I cement The aggregate samples came from the contractor's stockpile for the Van Buren project Grain-size distribution curves for this material are presented in Appendix B, Figure 1 Because of the limited time between the availability of the modified subbase samples and the start of construction, it was not possible to perform preconstruction wet-dry and freeze-thaw durability tests

A maximum dry density of 135 0 pcf was determined for both the 6 and 8 percent cement mixtures The optimum moisture content was 8.1% and 8 7% for the 6 and 8 percent cement mixtures, respectively Thus, cement content had no effect on maximum dry density and only a slight effect on the optimum water content Samples for the compressive strength tests were prepared in accordance with ASTM D 559 (AASHTO T 135), Wetting And Drying Tests Of Compacted Soil-Cement Mixtures (Appendix A) The prepared samples were cured for 7 days in a humid room then subjected to compression testing in accordance with ASTM D 1633, Compressive Strength Of Molded Soil-Cement Cylinders as discussed in Appendix A Additional discussion of the procedure is given in Nunan and Humphrey (1989) Compressive strength test results are presented in Figure 4 1

A 7-day compressive strength between 300 and 800 psi indicates a durable soil-cement base (PCA, 1971) As shown on Figure 4 1, compressive strength results at 6% cement are at the upper end of this range, while the results at 7% and 8% cement are above the upper end There are two other considerations (1) High compressive strengths may result in stabilized materials which act like a concrete pavement, creating reflective cracks in the upper asphaltic pavement For example, NYDOT used 8 to 10 percent cement in one of their stabilization projects which resulted in significant cracking (Chapter 2) So it is not desirable to use a cement content which is too high (2) The strength of field mixed and compacted soil cement is less than for laboratory samples Based on these considerations, it was reasoned that 6 percent cement would result in a strength in the middle of the recommended range and should be used in the experimental highway section

The specifications for the Van Buren field trial project allowed the contractor only four hours to mix and compact the soil-

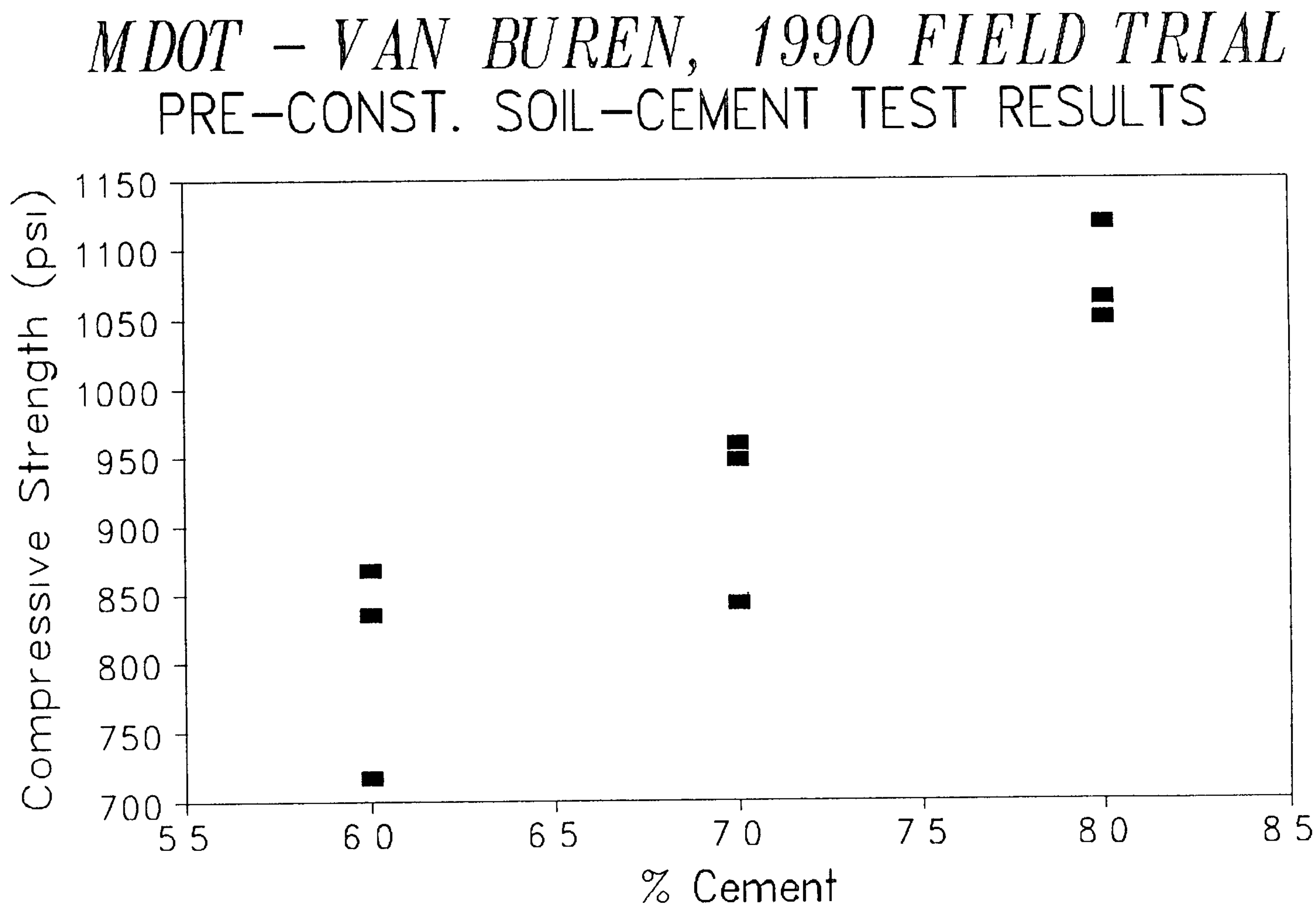

Figure 4 1 Preconstruction soil-cement compressive strength test results cement stabilized soil The specifications also indicated the acceptable mixing moisture conditions Therefore, the ability to determine material moisture contents rapidly was critical It was initially thought that the proprietary "Speedy" moisture test would help the contractor during construction of the soil-cement section by producing rapid moisture content results Thus, an attempt was made to correlate Speedy moisture results with oven-dry samples before construction began The Speedy moisture results were inconsistent, however The moisture content with the Speedy ranged between 0 7 percentage points below to 2 3 percentage points above those determined from oven dried samples The experimental nature of the project required more accurate and consistent results as were achieved by the nuclear gauge and drying the soil in a pan on a Coleman stove

CONSTRUCTION OF SOIL-CEMENT STABILIZED SECTION

Soil-Cement Mixing Operation

The contractor constructed the soil-cement stabilized section on 10 September 1990 The weather was sunny, warm, and dry The contractor placed the modified subbase aggregate in this experimental section as described in Chapter 3

The project specifications (MDOT, 1990) stipulated that the moisture content of the modified subbase aggregate at the time of cement application could not exceed the optimum moisture content of the soil-cement mixture The optimum water content of the 6 percent cement mixture was 8 1% Grab samples of the modified subbase aggregate were taken and the moisture content was determined using a Coleman stove to drive off the moisture By this method, moisture contents of 7 7%, 7 7%, and 8 3% were determined An MDOT technician also determined the aggregate moisture content using a nuclear density gauge The in-place modified subbase aggregate moisture content was 7 7% and 8 2% in two tests at different locations within the soil-cement section Therefore, the average moisture content resulting from both test methods conformed to the specifications and no adjustment was needed prior to adding the cement

To produce a 6% cement by weight, stabilized base mixture in a 6-inch lift, the contractor had to place 4 2 lbs of cement per square foot of roadway before mixing. In a cleared area adjacent to the highway, the contractor experimented with a 10 foot wide agricultural fertilizer spreader to determine whether or not it could be used to spread the Type I cement over the soil-cement section Unfortunately, it was determined that the maximum application rate that the fertilizer spreader was capable of was 0.15 lbs per square foot This meant that the contractor would have to make 28 passes over the section to achieve 4 2 lbs per square foot which was not practical Consequently, the contractor decided to place the cement with hand labor

Thus, the contractor had to place 4 bags of cement at stations every 3 7 feet The bags were equally spaced across the section The 3 7 foot stations were laid out and marked with orange paint

The contractor then used a front end loader to transport the cement, while laborers placed the bags across the section at each station As specified, the rate of cement application was reduced for the first and last 10 feet of the section by increasing the spacing of the bags This created a transition zone between adjacent sections

The specifications required that the contractor complete the spreading, mixing, and compaction of the soil-cement within 4 hours The contractor began the cement spreading operation at 2 30 pm after all the bags of cement were laid out The process began by first cutting open the bags, dumping the contents in a pile, and initially dispersing the cement with hand rakes After the initial spreading operation, the contractor spread the cement with a York brand spring tooth harrow pulled behind a John Deere 950 tractor This ingenious method produced a very uniform layer of cement over the entire 200 foot long by 24 foot wide section.

The contractor began mixing the cement and modified subbase aggregate at 3 20 pm For mixing, the contractor used a David Brown 990 tractor with a 5 foot wide roto-tiller attachment driven by a power take-off The roto-tiller was equipped with 20-inch diameter tines The depth and uniformity of mixing was examined after the contractor had made two passes At that point, it was obvious that the depth of mixing was insufficient and the cement was not uniformly distributed

The contractor was then directed to perform two more passes over the entire section Upon examination, it was found that the materials were uniformly mixed to the full depth after four passes It is possible that three passes may have worked It was also noted that the mixing process did not exceed the depth of the stabilized materials The lower subbase soils were in a very dense condition and contained relatively large size (6-inch minus) particles which could not be penetrated by the roto-tiller

Soil-Cement Compaction

The specifications required that the soil-cement be compacted to 97% of maximum dry density as determined by a field moisture-density test in accordance with AASHTO T134 (ASTM D 558) However, it was decided that it was appropriate to use the maximum dry density and optimum moisture content determined in a preconstruction laboratory moisture-density test performed in accordance with ASTM D 558 (Appendix A)

Compaction of the soil-cement base aggregate began at 4·10 pm The contractor used a Komatsu 605 A motor grader to level the surface of the treated soil After grading, the contractor used an Ingersoll-Rand SP 48 DD, 4 5-ton static drum weight, vibratory roller to compact the soil-cement It became apparent that the Model 48 compactor was inadequate to achieve the specified compaction level The contractor then switched to an Ingersoll-Rand Model SP 56 DD, 6-ton static drum weight, vibratory roller and successfully compacted the soil-cement mixture

Efficiency of the compaction process was monitored in a previously uncompacted section by observing the in-place density after the first, second, third, and fourth passes of the 6-ton roller An MDOT technician measured the in-place soil-cement density with a nuclear density gauge The density and post-treatment moisture content test results are presented in Table 4 1 As shown in the table, four passes of the compactor were necessary to achieve the specified compaction levels with the 6-ton vibratory roller

The specifications also required that the soil-cement moisture content be plus or minus 2% of the optimum water content of the soil-cement mixture All of the post-treatment moisture contents met the specifications Moisture content results also indicate that adding cement to the modified subbase aggregate and air drying during mixing and compaction only lowered the moisture content slightly. This can be seen by comparing the moisture contents determined by nuclear gauge before adding cement (7 7% and 8 2%) with those after compaction (6 9% to 7 8%). The contractor completed the compaction operation at 5·30 pm

Soil-Cement Fine Grading and Seal Rolling Operations

When the compaction process was complete, the contractor began fine grading with a Komatsu GD 605 A motor grader The compacted soil-cement grade was generally about 1 inch high The contractor graded the excess material to windrows just outside of the travel lanes and recompacted the surface The contractor cut slightly more than necessary at several locations, and consequently, the grade be-

Table 4 1 Soil-cement moisture and in-place density test results

STATION	NO. OF PASSES	MOISTURE CONTENT %	PERCENT COMPACTION
1028+50 Rt Lane	1	7 0	92 0
1028+50 Rt Lane	2	7 4	94 2
1028+50 Rt Lane	3	7 8	94 7
1028+50 Rt Lane	4	7 2	97 0
1029+10 Lt Lane	4 min	7 8	96 8
1029+50 Rt Lane	4 min.	7 6	98 4
1029+90 Rt Lane	4 min	6 9	96 8
1028+25 Lt Lane	--	8.2*	--
1028+75 Rt Lane	--	8 0*	--
1029+25 Lt Lane	--	8 9*	--
1029+75 Rt Lane	--	9 0*	--
--	--	7 7**	—

* These are water content test results after mixing, but before compaction of the soil-cement These tests were performed with a Coleman stove

** This water content was determined by a test performed with the nuclear density gauge before the compaction process came too low at these locations The contractor then brought additional material back from the windrow and recompacted the surface

It was observed that some of the material brought back from the windrow had inadequate cement content This resulted in a small amount of near surface material having insufficient cement content along the edge of the travel lane This occurred at those locations where it was necessary for the contractor to bring material back from the windrow to raise the grade This condition should be anticipated by more careful control of grade to prevent undercutting some areas and by keeping excess material within the outer limit of the stabilized width until final grade is achieved Then, excess material can be bladed beyond the limits of the stabilized width The fine grading and compaction operation was complete at 6 15 pm

The contractor performed the final surface seal-rolling when the fine grading operation was complete The surface of the soil-cement base was moistened with water pumped from a water truck through a fire hose with a fine spray The contractor then seal rolled the surface with three passes of a 9-ton BROS pneumatic rubber-tired roller When rolling was complete, the contractor remoistened the surface to help preserve moisture in the soil-cement overnight This operation was complete at 6 40 pm

As noted earlier, the specifications allowed the contractor 4 hours to complete mixing, grading, and compaction of the soil-cement The specifications also required the contractor to complete compaction and finishing within 2 hours after that operation began Overall, the contractor took only 10 minutes longer than the allotted 4 hours to complete the soil-cement section construction However, the compaction process began at 4 10 pm and continued through the finish operation completed at 6 40 pm Compaction and finishing, therefore, took 30 minutes longer than allowed in the specification The contractor would have successfully completed the compaction and finishing operations within the time limits if the large compactor was used at the beginning of the compaction process The contractor lost valuable time waiting for the large compactor to be brought from a distant part of the project

There may not be any significant long-term detrimental effect since the operation was essentially complete within 4 hours However, the effect of breaking the soil-cement bonds during compaction and reforming the bonds as the mixture continues to cure is not immediately apparent Possibly the worst consequence of the slowed compaction process is that the soil-cement will have slightly lower compressive strength. Additionally, the contractor had to overcome the bond strength, which increased as the concrete cured, to compact the soil-cement to the specified compaction level This means that probably one or two more passes with the compaction equipment were required to achieve the specified compaction level

Bituminous Curing Material Placement

At about 10 00 am the following morning, the contractor remoistened the finished soil-cement surface with a water truck and fire hose as previously described The contractor performed this operation to fill surface voids with water and thus prevent excessive penetration of the bituminous curing material The contractor applied MS4 emulsified asphalt to the soil-cement surface immediately after moistening the surface The asphalt temperature at the time of application was about 150 degrees Fahrenheit The contractor applied the emulsified asphalt at a rate of about 0 35 gallon per square yard

Placement of the bituminous curing material completed the construction of the soil-cement section The soil-cement cured for 7-1/2 days before traffic was allowed to travel over the section The soil-cement section was paved 9 days after it was completed

Final Observations Before Paving

The soil-cement section was inspected just before the contractor began paving on 19 September 1990 Traffic was allowed to travel over the experimental section for about 36 hours before the paving operation began No surface raveling was observed in the soil-cement section wheel paths There were, however, 1/8-inch deep pockmark depressions where the bituminous seal coat followed the soil-cement surface The pockmarks ranged from 1 inch to 5 inches in diameter It is felt that these pockmarks will not affect performance of the section Chapter 3 discusses the paving operations and subsequent Road Rater pavement deflection measurements

Aggregate Sampling and Field Compacted Soil-Cement Cylinders

Modified subbase aggregate was collected from the soil-cement section before the contractor mixed in the cement The samples were used for post-construction laboratory grain size, moisture-density, and numerous soil-cement tests

12 soil-cement cylinders were compacted for use in laboratory compression, freeze-thaw, and wet-dry tests The mixed soil-cement was sampled for compacting into the cylinders from STA 1028+25, Lt Lane, STA 1028+75, Rt Lane, STA 1029+25, Lt Lane, and STA 1029+75, Rt Lane, after the contractor completed the mixing operation The cylinders were made while the contractor graded and compacted the soil-cement stabilized section.

The samples were compacted in general conformance with ASTM D 559 (AASHTO T 135), Wetting And Drying Tests Of Soil-Cement Mixtures as discussed in Appendix A. It was not possible to perform the oversize correction as described in the ASTM standard However, the field procedure was adapted by removing the particle sizes with a diameter greater than 3/4-inch determined by visual inspection Furthermore, the gravel size particles for the field generated cylinders were not screened out, soaked, and recombined as prescribed in the ASTM laboratory procedure because the stabilized material was mixed in-place The average dry unit weight of the field compacted soil-cement cylinders was 126 0 pcf

The compacted cylinders were cured inside sealed plastic freezer bags at ambient temperature until placed in the University of Maine humid room The samples were protected from direct sunlight Nighttime temperatures were relatively cool and dropped to the mid-thirties one evening After two days of field curing, the soil-cement cylinders were transported to the University of Maine, protecting them against vibration by careful packing The cylinders were removed from the plastic bags and placed them in the humid room at the university for the remainder of the seven day curing period

POST-CONSTRUCTION LABORATORY TESTING

Grain Size Analysis Tests

The modified subbase aggregate in the soil-cement section was sampled before the contractor mixed it with cement Sieve analysis tests were conducted on this material and the results are presented in Appendix B, Figure 2 The test results show a slight increase (4 8% vs 4 5%) in the fines content as compared to the fines content of the aggregate sampled from the stockpile Note that it was impossible to perform sieve analysis tests of the stabilized aggregate in this section

Soil-Cement Compression Tests

The field compacted soil-cement cylinders were subjected to compression, freeze-thaw, and wet-dry tests after the 7-day curing period was complete To replicate the soil-cement field mixture for the laboratory prepared samples, it was first necessary to determine the average cement content Remains of the field compacted wet-dry and freeze-thaw cylinders were used for cement content tests (Appendix A) As a check on the accuracy of the procedure, the remains of a 6% cement preconstruction cylinder were also tested Since 4 6% cement was measured in that cement content test, this casts doubt on the accuracy of the procedure Nevertheless, the cement content results were used as the basis for post-construction laboratory testing Table 4 2 presents the results of the cement content tests The tests of the wet-dry and freeze-thaw field compacted cylinders ranged between 5 4% and 7 5% cement by weight of soil with an average of 6 4% cement Because of round-off differences in the initial average percent cement calculation, 6 3% cement was used in the laboratory-made cylinders The moisture content for preparation of post-construction laboratory cylinders was determined from oven-dried field samples The results of these water content tests are presented in Table 4.4 The average moisture content of 7 4% from these tests was used to make-up the laboratory cylinders Compression test results for the 6% cement preconstruction cylinders, field compacted cylinders, and two sets of post-construction laboratory prepared cylinders are presented in Table 4 3

As previously mentioned, a 7-day compressive strength between 300 and 800 psi will provide a durable soil-cement base (PCA, 1971) All of the field compacted cylinders exhibited 7-day compressive strengths within this range

The field compacted freeze-thaw and wet-dry cylinders were also subjected to compression testing after 35 or 37 days of curing The tests were performed after completion of 12 freeze-thaw or wet-dry cycles as prescribed in those respective tests These tests provide an indication of the soil-cement's residual compressive strength The results given in Table 4 3 show that the compressive strength of the field compacted freeze-thaw and wet-dry soil-cement cylinders

Table 4 2 Cement content test results

Sample Description		Percent Cement
STA 1028+75R, Wet-Dry Cylinder Remains		5 4
STA 1029+25L, Wet-Dry Cylinder Remains		7 2
STA 1029+75R, Wet-Dry Cylinder Remains		7 0
STA 1028+25L, Freeze-Thaw Cylinder Remains		7 5
STA 1028+75R, Freeze-Thaw Cylinder Remains		6 1
STA 1029+25L, Freeze-Thaw Cylinder Remains		5 7
STA 1029+75R, Freeze-Thaw Cylinder Remains		6 3
	Avg	6 4
6% Cement Preconstruction Cylinder Remains		4 6

also meets the PCA criteria and is somewhat higher than the 7-day compressive strength, probably due to the longer curing time

The 7-day compressive strength of the field compacted cylinders is about 200 to 400 psi lower than the preconstruction and post-construction laboratory cylinders One potential reason for this

Table 4 3 SOIL-CEMENT COMPRESSION TEST RESULTS (psi)

Station	Preconstruction Cylinders	Field Compacted Cylinders			Post-Construction Cylinders: 3/4-Inch Minus Mat'l				Post-Construction Cylinders: Oversize Correction			
	7-Day	7-Day	F/T*	W/D*	7-Day	28-Day	F/T**	W/D**	7-Day	28-Day	F/T**	W/D**
	835											
	716											
	867											
Avg	806											
1028+25L		551	507	610								
1028+75R		390	416	549								
1029+25L		485	668	664								
1029+75R		517	591	517								
Avg		486	546	585								
					829	1044	959	1325				
					792	1140	1170	1144				
					889	1106	1068	1122				
Avg					837	1097	1066	1197				
									971	1186	1092	1245
									857	915	1138	1206
									933	1202	961	1436
Avg									920	1101	1064	1296

F/T = Freeze-Thaw Cylinders
W/D = Wet-Dry Cylinders
* 35 Day Cylinders
** 37 Day Cylinders

Preconstruction Cylinders Were Made With 6% Cement
Field Cylinders Contained An Average Of 6 3% Cement
Post-Construction Cylinders Were Made With 6 3% Cement

disparity was oversize correction of the aggregate ASTM D 559 (AASHTO T 135) requires that the soil used to make soil-cement cylinders be corrected for oversize particles Specifically, particle sizes between 3/4-inch and 3-inch diameter must be replaced with an equal weight of No 4 (approximately 3/16-inch) to 3/4-inch particles However, it was not possible to perform this correction

Table 4 4 Field sample water content results

Station	Water Content %
1028+25 R	6 8
1028+25 L	6 6
1028+75 R	7 9
1028+75 L	7 6
1029+25 R	7 7
1029+25 L	7 4
1029+75 R	7 9
1029+75 L	7 0
Avg	7 4

on the field soil-cement mixture Therefore, two iterations of laboratory testing were performed to explore the difference in mixture one with samples corrected for oversize particles, and another with 3/4-inch minus aggregate only In both cases, the aggregate larger than No 4 was soaked before mixing The results of these tests are also given in Table 4 3

It was determined that laboratory cylinders with more gravel size particles (samples corrected for oversize particles) had an average compressive strength about 100 psi higher than the lab cylinders containing 3/4-inch minus aggregate only This indicates that some of the difference in strength may be attributed to oversize correction, but cannot account for the entire 200 psi to 400 psi difference There was almost no difference in the 28-day freeze-thaw or wet-dry strengths for the 3/4-inch minus and oversize corrected samples

One observation may be the key to field and laboratory strength differences In its natural excavated condition, the aggregate had a coating of fine silt The silt coating may limit the compressive strength of the mixture by inhibiting the bond between the aggregate and the cement This explanation is even more plausible when considering the ASTM D 559 laboratory procedure for preparing soil-cement mixtures. ASTM D 559 requires that the No. 4 to 3/4-inch size particles be soaked and surface dried before recombining this fraction with the sand fraction and cement This produces a very clean aggregate surface, to which cement may now form a much stronger bond

Another possible explanation for lower compressive strengths of field compacted cylinders is the compaction mold reaction with the ground surface A 1-inch thick steel plate was used for a reaction block on the ground surface In the laboratory, the soil-cement cylinders are compacted on a concrete floor The ground surface may absorb more of the hammer energy than the concrete floor, imparting less energy to the soil-cement in the mold and resulting in a lower density for the field samples The average dry unit weight of the field compacted samples was 126 0 pcf for the 12 field compacted cylinders compared to an average of 130 9 pcf for the 24 laboratory compacted cylinders

Several other factors may have influenced the compressive strength results to a lesser degree (1) The field compacted samples cured at ambient temperature in plastic bags to preserve moisture The laboratory compacted samples were cured in a humid room at constant temperature This ready access to moisture and high temperature could tend to yield higher strengths (2) There is a longer period between mixing and compaction in the field as compared to the laboratory process where the sample is mixed and immediately compacted into a mold Thus, more of the cements reactivity is lost prior to compaction in the field (3) Mixing the soil and cement in the field is probably less efficient than the laboratory procedure which requires mixing the cement with the sand fraction before adding the coarse aggregate

Freeze-Thaw and Wet-Dry Soil-Cement Tests

One measure of soil-cement durability is the percent soil-cement loss from freeze-thaw and wet-dry tests (Appendix A) For AASHTO A-1 soils, the maximum allowable soil-cement loss after 12 cycles of wet-dry or freeze-thaw testing is 14 percent (PCA 1971) Table 4 5 presents the results of freeze-thaw and wet-dry tests performed on fıeld compacted and post-constructıon soıl-cement cylınders All of the results meet the PCA crıterıa

The percent soıl-cement loss for laboratory samples, both the

Table 4 5 Percent soıl-cement loss from freeze-thaw and wet-dry tests

Sample Descrıptıon	Percent Loss
STA 1028+25L, Fıeld Compacted Wet-Dry Cylınder	8 5
STA 1028+75R, Fıeld Compacted Wet-Dry Cylınder	10 0
STA 1029+25L, Fıeld Compacted Wet-Dry Cylınder	6 2
STA 1029+75R, Fıeld Compacted Wet-Dry Cylınder	7 8
STA 1028+25L, Fıeld Compacted Freeze-Thaw Cylınder	3 7
STA 1028+75R, Fıeld Compacted Freeze-Thaw Cylınder	3 4
STA 1029+25L, Fıeld Compacted Freeze-Thaw Cylınder	3 3
STA 1029+75R, Fıeld Compacted Freeze-Thaw Cylınder	5 0
Lab Compacted, 3/4-Inch Mat'l , Wet-Dry Cylınder	0.4
Lab Compacted, 3/4-Inch Mat'l , Wet-Dry Cylınder	0 5
Lab Compacted, 3/4-Inch Mat'l , Wet-Dry Cylınder	0 1
Lab Compacted, Oversıze Corr., Wet-Dry Cylınder	0 8
Lab Compacted, Oversıze Corr , Wet-Dry Cylınder	1 0
Lab Compacted, Oversıze Corr , Wet-Dry Cylınder	0 5
Lab Compacted, 3/4-Inch Mat'l., Freeze-Thaw Cylınder	0 3
Lab Compacted, 3/4-Inch Mat'l , Freeze-Thaw Cylınder	0 4
Lab Compacted, 3/4-Inch Mat'l , Freeze-Thaw Cylınder	0 2
Lab Compacted, Oversıze Corr , Freeze-Thaw Cylınder	0 5
Lab Compacted, Oversıze Corr , Freeze-Thaw Cylınder	0 4
Lab Compacted, Oversıze Corr , Freeze-Thaw Cylınder	0 3

3/4-ınch mınus mıxture and the mıxture corrected for oversıze partıcles, were very small (1% or less) and near equal for each mıxture Based on lımıted analysıs, thıs ıs another ındıcatıon that the 3/4-ınch fıeld mıxture and laboratory oversıze corrected soıl-cement mıxture should not produce widely disparate strength and durability results

The field compacted cylinders experienced significantly greater soil-cement loss This may also be the result of weaker bond strengths The fine silt coating prevents the aggregate from bonding with the cement This makes it easier for the aggregate to break away from the cylinders during the brushing procedure which is part of the freeze-thaw and wet-dry tests All of the test results, however, still met the above described PCA criteria for percent soil-cement loss

SUMMARY

Based on preconstruction compressive strength tests, a cement content of 6 percent was chosen for the soil-cement stabilized section This was expected to have compressive strengths in the range recommended by PCA yet not be so strong as to cause excessive cracking of the overlying asphaltic concrete course

The test section was constructed in general conformance with the construction specifications Four passes of a 20 inch diameter roto-tiller were required to achieve a uniform distribution of cement throughout the 6 inch thickness of the stabilized base course The contractor could not obtain the specified density with several passes of a smooth drum vibratory roller with a static drum weight of 4 5 tons However, a larger 6-ton roller could obtain the specified density with 4 passes It was felt that the additional compactive energy was needed to overcome the cement bonds which begin to form as soon at the cement is mixed with the aggregate Compaction was delayed while the larger roller was brought to the test section This caused the time limits on mixing and compaction to be exceeded slightly Care should be used in final grading to be sure that aggregate with insufficient cement is not brought from the windrows at the sides of the test section and mixed with the treated soil

After 7-1/2 days of curing, traffic was allowed over the soil-cement stabilized section for about 36 hours No visible surface raveling occurred as a result of this traffic Thus, after curing, traffic may be allowed to use the soil-cement stabilized base for short periods prior to paving.

Samples of the cement stabilized soil were taken as soon as the contractor completed mixing operations Then while the contractor was compacting the stabilized section, the field-mixed soil cement samples were used to prepare cylinders for compression, wet-dry, and freeze-thaw tests Samples of treated and untreated aggregate were also collected for grain size and water content tests Large samples of untreated aggregate were also obtained for making laboratory compacted specimens

Laboratory sieve analysis tests showed a slight increase in the fines content after loading, hauling, and grading the modified subbase aggregate placed in this section Cement content tests on samples of treated aggregate showed 6 4 percent cement, although the accuracy of this test may be questionable The average field water content was 7 4 percent Based on these values, laboratory mixed samples were prepared for compression, wet-dry, and freeze-thaw tests The actual cement content of these specimens was 6 3 percent

Field mixed samples had a lower 7-day compressive strength than laboratory mixed samples by nearly a factor of 2, although, the strength for both field and laboratory mixed samples fell within the range recommended by PCA (300 to 800 psi) for durable soil-cement mixtures Furthermore, the percent loss in the wet-dry and freeze-thaw tests were significantly higher for the field mixed samples (3 3 to 10.0 percent) than the laboratory mixed samples (0 1 to 1 0 percent) This difference is so large that wet-dry and freeze-thaw tests seem to have limited value for predicting field durability

There are several reasons for the large differences noted above (1) The plus No 4 particles were soaked in the laboratory prior to sample preparation as specified in AASHTO T 135 (ASTM D 559) This removed a coating of silt which covered the surface of the aggregate and improved the bond between the aggregate and the cement (2) An oversize correction was not made in the field compacted samples (3) The laboratory samples were cured at higher humidity and temperature than the field samples

(4) The cement was probably more uniformly mixed with the aggregate in the laboratory prepared samples (5) The delay between mixing and compaction was shorter in the laboratory than in the field Thus, more of the cement's reactivity was available in the lab (6) Because of the compaction procedure used in the field, these samples had a lower density It is felt that the first reason is the most important Thus, the possibility of using unsoaked plus No 4 aggregate should be investigated All but the last reason will occur in most laboratory versus field situations Thus, it is prudent to choose the cement content to result in a strength near the upper end of the PCA limit (i e , 800 psi) for laboratory mixed samples

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CHAPTER 5

ASPHALT STABILIZED BASE

PRECONSTRUCTION LABORATORY TESTING

The amount of MS-4 emulsified asphalt needed to stabilize the modified subbase aggregate and compaction control data had to be determined prior to construction of the experimental section In addition, there was a need to understand what effects moisture contents greater than the recommended 3% had on stability results Therefore, Marshall stability tests were conducted on asphalt stabilized base materials using 4 5%, 5 5%, and 6 5% asphalt by weight, while varying the moisture content. The tests were performed on modified subbase aggregate samples mixed with MS-4 emulsified asphalt The aggregate samples came from the contractor's stockpile for the Van Buren project Grain-size distribution curves for this material are presented in Appendix B, Figure 1.

The samples were prepared and tested in accordance with the Modified Marshall Test procedures as discussed in Appendix A Table 5 1 presents the average preconstruction Marshall test results The Marshall stability results from the preconstruction tests versus asphalt content are shown on Figure 5 1 A complete summary of the preconstruction Marshall stability test results is presented in Appendix D, Table 1

Table 5 1 Average Marshall stability and flow index results of preconstruction laboratory tests

Asphalt Content (%)	Water Content At Mixing (%)	Unit Weight (pcf)	Marshall Stability (lbs)	Flow Index (0 01")
4 5	3 0	137 8	1164	9 9
5 5	3 0	139 0	928	5 8
5 5	6 0	133 6	301	13 6
5 5	9 0	131 1	138	21 6
6 5	3 0	137 1	617	7 9

The test results show that Marshall stability values decreased as the asphalt content was increased from 4 5% to 6 5% while maintaining the moisture content at 3% For medium traffic, a minimum stability of 500 lbs is specified as the Marshall design criteria (The Asphalt Institute, 1974) This criteria was met for all samples with 3% water content at mixing. However, the highest stability was obtained with 4 5% asphalt Therefore, based on the above criteria and the preconstruction Marshall stability test results, 4 5% asphalt was specified for the field trial

The results also indicated that Marshall stability values decreased as the moisture content was increased from 3% to 9% while maintaining the asphalt content at 5 5% For moisture contents of 6%

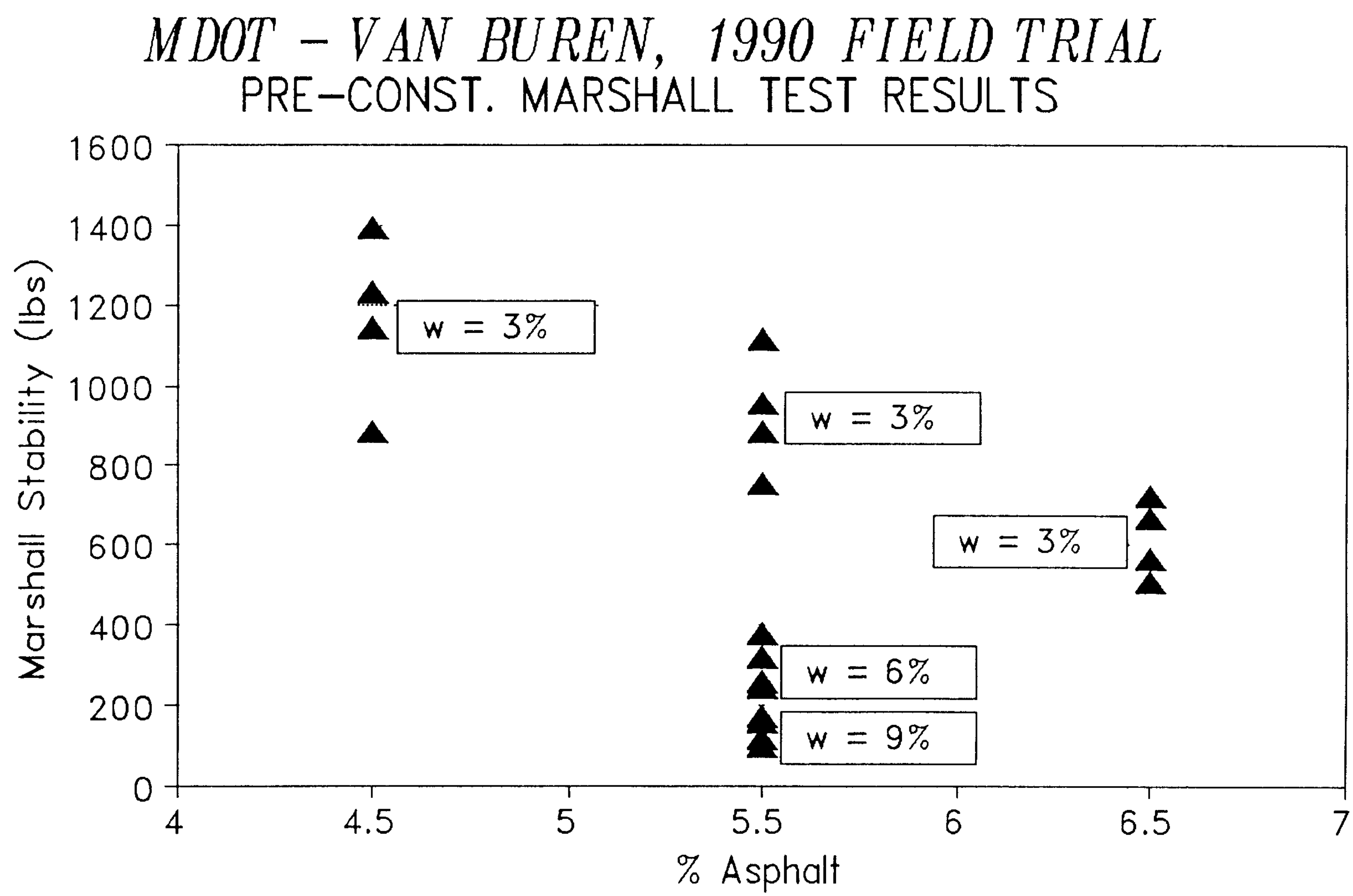

Figure 5 1 Preconstruction Marshall stability test results and 9%, the stability was less than the recommended minimum of 500 lb Based on these results, it was estimated that at 4 5% asphalt and 6% moisture content the stability number would be about 500 lb Thus, it was decided that the maximum acceptable field water content would be 5% at an asphalt content of 4 5%

CONSTRUCTION OF ASPHALT STABILIZED SECTION

Emulsified Asphalt Mixing Operation

The contractor constructed the asphalt stabilized section on 11 September 1990 The weather in early morning was partly cloudy but soon cleared to sunny, warm, and dry conditions The contractor placed the modified subbase aggregate in this section of the experimental highway the previous day as described in Chapter 3 After placement, the contractor had compacted the modified subbase aggregate with an Ingersoll-Rand Model SP 48 DD compactor in the static drum mode Therefore, the contractor's initial construction task was to till-up the aggregate For this operation, the contractor used the tractor and roto-tiller previously described for soil-cement mixing

The modified subbase aggregate had a moisture content of about 8% when it was brought to the construction site from the borrow pit Since the project specifications stipulated that the asphalt was not to be applied when the moisture content of the aggregate was greater than 3%, aeration was necessary to reduce the moisture content of the aggregate As discussed above, it was estimated that 5% moisture would result in a mixture with a Marshall stability that was above the minimum value The contractor was able to reduce the moisture content to about 6 6% by aerating the aggregate for about five hours with the roto-tiller

The contractor was directed to apply the asphalt at this time However, because the aggregate moisture content did not meet the specification or the estimate for a stable mix, it was stipulated that the contractor had to re-aerate the asphalt stabilized aggregate on the next available sunny day to further reduce the moisture content of the mix

The contractor applied the MS-4 emulsified asphalt with an E D Etnyre & Co , Model BX, Style RE asphalt distributor truck with a tank capacity of 1314 gallons. To prevent excess run-off of the freshly placed asphalt, the contractor made four applications of emulsified asphalt and tilled it into the aggregate after each application Table 5 2 summarizes the application rates and temperatures The application rates were corrected to standard temperature in accordance with MDOT specification 702 06

The contractor mixed the MS-4 asphalt into the aggregate with one pass of the tiller after the first and third applications, two passes of the tiller after the second application, and five passes of the tiller after the final asphalt application At this point, it was noted that the field mixture had a visual appearance similar to the preconstruction laboratory mixture The distribution and mixing of asphalt took approximately 1-1/4 hours When mixing was complete, the contractor graded the stabilized aggregate with a Komatsu GD 605A motor grader

Table 5 2 Summary asphalt application rates and temperatures

Application		Application Rate (gal/yd^2)	Temperature (Farenheit)	Time
First		1 00	150	2 40 pm
Second		1 01	150	3 00 pm
Third		0 81	140	3 30 pm
Fourth		0 57	140	3 50 pm
	Total	3 39		

Asphalt Stabilized Base Compaction And Fine Grading Operations

There was concern that compaction would be a problem due to the moisture content of the mix Therefore, the contractor was directed to trial compact a small area of the asphalt stabilized section The contractor compacted the trial area with an Ingersoll-Rand Model SP 56 DD vibratory roller The contractor easily achieved 100% of maximum dry density which was much greater than the specified compaction of 93% The maximum dry density was based on preconstruction Marshall sample unit weights

All of the field in-place density and moisture content test results are presented in Table 5 3 Note that the nuclear density gauge registers asphalt as moisture Accounting for the asphalt and

Table 5 3 Asphalt stabilized base moisture content and in-place density test results

STATION	NO. OF PASSES	SOIL MOISTURE %	TOTAL MOISTURE %	PERCENT COMPACTION
--		6 5*		--
--		6 9*		--
1032+25 Lt Lane	0	6 5	8 0	81 3
1033+47 Lt Lane	2	6 5	8 0	100 2
1033+47 Lt Lane	3	6 8	8 3	100 8
1033+47 Lt Lane	4	6 8	8 3	101 8
1033+75 Rt Lane**	--		5 5	98 6
1033+20 Lt Lane**	--		6 3	100 9
1032+92 Rt Lane**	--		4 8	103 0
1032+50 Lt Lane**	--		5 4	103.0

* These are water content test results of random samples of the modified subbase aggregate before treatment These tests were performed by drying the samples with a Coleman stove

** These tests performed after re-aeration and compaction water contained in the emulsion, the soil moisture ranged between 6 5% and 6 8% Since the emulsified asphalt is 35% water, the actual total water content of the mixture ranged between 8 0% and 8 3%

A comparison of compaction energy per cubic foot of volume between the Marshall test and the Modified Proctor test shows that Marshall samples are subjected to nearly twice the compaction energy (103,448 ft-lb/ft^3 vs 56,250 ft-lb/ft^3) of Modified Proctor samples Thus, densities in the range of 100% of Marshall density, as in the trial compaction area, represent a very dense matrix

As expected with the high water content, the asphalt stabilized aggregate had insufficient stability This was evidenced by the ability to make impressions on the surface of the stabilized material with the heel of a boot and the rubber tires on the vibratory roller The remainder of the asphalt stabilized section was brought to approximate grade and static rolled only This would make it easier for the contractor to till-up and aerate the stabilized aggregate again on the next sunny day.

Two days later on 13 September 1990, the contractor re-aerated the asphalt stabilized aggregate with tractor and tiller for about five hours The weather was partly cloudy, warm, and dry After re-aeration, the asphalt stabilized base was graded and compacted with equipment previously described Compaction was achieved with two or three passes of the vibratory roller The results of density tests in the re-aerated and compacted asphalt stabilized material are also presented in Table 5 3

The specifications required a minimum compaction level of 93% of Marshall density as determined by laboratory Marshall tests All of the density tests performed in the re-aerated asphalt stabilized base conformed to the specifications The moisture content results in Table 5 3 show that the contractor was able to reduce the asphalt stabilized aggregate moisture to a water content of about 5 5% Considering the water added with the emulsion, this is equivalent to an initial aggregate water content of about 4%

After re-aeration and compaction, stability of the asphalt stabilized base was significantly improved The rubber tires on the vibratory roller no longer made an impression in the asphalt stabilized materials The asphalt stabilized base cured for a total of two days at the higher water content and four days at the lower water content before traffic was allowed over the section

Final Observations Before Paving

The asphalt stabilized section was inspected just before the contractor began paving on 19 September 1990 The project resident engineer noted that the surface of the asphalt stabilized base had occasional cracking in a honeycomb pattern similar to a desiccated clay, but with much larger interlocking honeycombs The largest crack noted was about 1 mm wide

Traffic was allowed to travel over the experimental section for about 36 hours before the paving operation began. It was observed that as more traffic used the section, the surface cracks became less and less frequent and those remaining became smaller in width. Slight raveling was also noted in the wheel paths in the northbound lane which had a maximum depth slightly less than 1/2-inch. This indicates a surface coat of MS-4 emulsified asphalt may be needed if the completed section will be subjected to several days of traffic before paving.

No apparent surface deflections were observed as large, heavily loaded pulp trucks and potato trucks traveled over the section. Thus, the stability of the asphalt stabilized base now appeared quite good. The asphalt stabilized section was paved 7 days after it was constructed. Chapter 3 discusses the paving operations and subsequent Road Rater pavement deflection measurements.

Aggregate Sampling and Field Compacted Marshall Samples

Modified subbase aggregate samples were collected from the asphalt stabilized section before the contractor distributed the emulsified asphalt. The samples were used for post-construction laboratory grain size and Marshall stability tests.

12 Marshall specimens were field compacted for later testing in the laboratory. Samples of the field mixed asphalt stabilized aggregate were obtained from STA. 1032+25, Lt Lane, STA. 1032+75, Rt Lane, STA. 1033+25, Lt Lane, and STA. 1033+75, Rt Lane, after the contractor completed the mixing operation. The specimens were com-

pacted while the contractor compacted the asphalt stabilized section

The samples were compacted in general conformance with the Modified Marshall test procedure (Appendix A) However, the asphalt stabilized aggregate was not sieved through a 3/4-inch screen before compacting the Marshall samples Instead, the particle sizes with a diameter greater than 3/4-inch were determined by visual inspection and removed

The compacted samples were allowed to cure at ambient temperature and humidity while the remaining test section was constructed The samples were protected from direct sunlight Nighttime temperatures were relatively cool and dropped to the mid-thirties one evening After one day of field curing, the Marshall samples were transported to the University of Maine, protecting them against vibration by careful packing The Marshall samples were allowed to cure at room temperature and humidity for the remainder of the 9-day curing period

POST-CONSTRUCTION LABORATORY TESTING

Grain Size Analysis Tests

Samples of modified subbase aggregate were collected before the contractor mixed in the emulsified asphalt Sieve analysis test results for this aggregate indicate that loading, hauling, grading, and static rolling the modified subbase aggregate increased the fines content 2% when compared to stockpile samples of the same aggregate

(See Appendix B, Figures 1 and 2)

MDOT performed grain size analysis on samples brought to their lab for asphalt extraction tests The results of these grain size tests are presented in Appendix B, Figures 3 and 4 Four samples were the remains of Marshall stability test samples made from field mixed materials after the first mixing and the two others were random grab samples of the re-aerated asphalt stabilized mixture

Sieve analysis of the random grab samples collected after re-aeration showed an additional 2% greater fines content than the loaded, hauled and graded aggregate Since this material was only static rolled before re-aeration, it was concluded that mixing with the roto-tiller caused this additional increase in fines This section was subjected to more passes with the roto-tiller than the other test sections because of the need to reduce the water content of the aggregate

The Marshall sample remains give an indication of the effects of compaction on the asphalt stabilized materials However, these samples had most of the particle sizes larger than 3/4-inch diameter removed before compaction Gradation analysis showed that these samples had 100% of their particle sizes finer than 1-inch in diameter Thus, a correction is needed to allow comparison with the in-place material

The asphalt stabilized aggregate samples after mixing contained about 15% to 20% particle sizes greater than 1-inch diameter (Appendix B, Figure 3) Therefore, dividing the fines content from the tested Marshall samples by 1 15 and 1 20 yields estimates of the fines contents of Marshall samples which would include particle sizes greater than 1-inch in diameter Based on the limited data, the resulting estimates indicate that compaction of the asphalt stabilized aggregate in the Marshall test might account for 1 4% increase in fines content in addition to that produced by aggregate placement and the first mixing

Marshall Stability Tests

The field compacted Marshall samples were subjected to stability tests (Appendix A) after curing for 9 days Marshall samples were also compacted from the re-aerated asphalt stabilized aggregate that was collected and placed in sealed plastic bags after the contractor re-tilled the section The re-aerated samples were stored for about one month before compacting the Marshall samples in the laboratory.

To replicate the asphalt stabilized base mixture for the laboratory prepared samples, the average asphalt content must first be determined MDOT performed asphalt extraction tests, in accordance with ASTM D 2172 as discussed in Appendix A, on remains of four field compacted Marshall samples and two grab samples of the re-aerated stabilized aggregate The average asphalt content resulting from these tests was 4 5%, the target asphalt content Note that this is percent by weight of dry soil The laboratory compacted Marshall samples were cured for 10 days before testing

Moisture contents determined by nuclear density gauge show that the soil moisture content at the time of asphalt application was 6 6% after subtracting the water in the emulsion The contractor was later able to reduce the soil moisture to approximately 4 0% by reaerating the stabilized mixture This provided two opportunities to examine the predictability of stabilized aggregate strength Thus, laboratory samples were made with both the originally mixed soil moisture content of 6 6% and the re-dried soil moisture content of 4 0%

For comparison, average Marshall stability test results for the 4 5% asphalt preconstruction samples, field compacted samples, and post-construction samples are presented in Table 5.4. The average flow index values and unit weights are also presented in Table 5 4 All of the field compacted and post-construction Marshall stability, flow index, and unit weight results are presented in Appendix D, Tables 2 and 3

For medium traffic, a minimum stability of 500 lbs is specified as the Marshall design criteria (The Asphalt Institute, 1974) All of the field compacted and post-construction lab compacted Marshall samples met that criteria The Marshall stability results varied less than 10% between field and lab samples This indicates that the laboratory stability tests can be used to predict stability of field mixed material Likewise, the flow index of the

Table 5 4 AVERAGE MARSHALL STABILITY[a], FLOW INDEX[b], AND UNIT WEIGHT[c]

	Preconstruction Lab Samples			Field Compacted Samples			Post Construction Lab Samples: Re-Aerated Field Samples			Post Construction Lab Samples: Lab Mixed Samples		
Moisture At Mixing (%)[d]	Stability	Flow	Unit Weight	Stability	Flow	Unit Weight	Stability	Flow	Unit Weight	Stability	Flow	Unit Weight
3 0	1164	9 9	137 8									
6 6[e]				683	13 4	134 0				751	13 2	137 0
4 0[f]							1440	4 3	140 7	1433	5 4	141 8

a (LBS)
b (0 01's inch)
c (PCF)
d Soil Moisture Only
e Average Soil Moisture Content (Excluding Emulsion) During Initial Construction
f Average Soil Moisture Content (Excluding Emulsion) After Re-Aeration

NOTES

1) All of the laboratory generated samples contain 4 5% asphalt by weight of aggregate
2) The field-generated samples contain an average of 4 5% asphalt by weight of aggregate as determined by MDOT asphalt extraction tests

field and laboratory mixed samples were similar

It is believed that the slight difference in unit weight between the field compacted and lab compacted samples at the same water content may be attributed to material consistency at the time of compaction As described in the test procedures in Appendix A, the same compaction equipment was used in the field as in the lab However, the lab prepared asphalt/aggregate mixtures were typically hotter This allowed greater densification but the effect on the Marshall stability number was negligible

SUMMARY

Preconstruction laboratory tests were performed to determine the asphalt content that would produce the highest Marshall stability Marshall tests were conducted using modified subbase aggregate obtained from the contractor's stockpile The water content of the aggregate was adjusted to 3% and tests were conducted with 4 5%, 5 5%, and 6 5% MS4 asphalt based on the dry weight of soil An asphalt content of 4 5% gave the highest stability (1164 lb) This was well in excess of the 500 lb minimum recommended by the Asphalt Institute, so this asphalt content was used for the field trial

The effect of higher initial water content of the aggregate was investigated for samples with an asphalt content of 5 5% The Marshall stability of samples with 6% and 9% initial water content was less than the Asphalt Institute minimum showing the large effect that the initial aggregate water content has on Marshall stability It was estimated that at an asphalt content of 4 5% and initial aggregate water contents below 5% would produce a Marshall stability in excess of the Asphalt Institute minimum Thus, an initial aggregate water content of 5% was deemed to be the maximum allowable in the field

The asphalt stabilized test section was constructed in general conformance with the construction specifications However, it was only possible to reduce the initial aggregate water content to 6 6% The contractor was allowed to apply the asphalt at this time provided the test section was re-aerated on the next sunny day to further reduce the moisture content of the mix.

The asphalt was applied to the modified subbase aggregate in four applications and tilled into the aggregate after each application with one or two passes of the roto-tiller. After the final application, five passes of the roto-tiller were needed to uniformly mix the asphalt with the aggregate A trial area was compacted with a 6-ton static drum weight, vibratory, smooth drum roller A density of 100% of the laboratory maximum was achieved with only two passes However, because of the high water content, the compacted asphalt stabilized aggregate had insufficient stability

Two days later the contractor re-aerated the asphalt stabilized aggregate with the roto-tiller for about 5 hours Subtracting the water added with the emulsified MS-4 asphalt, the water content of the aggregate was reduced to about 4% After grading and compaction, the section now had significantly improved stability Thus, had it been possible to reduced the initial aggregate water content to 4%, the compacted mix would have had adequate stability without the need to re-aerate the mix to further reduce the water content

Traffic was allowed to travel over the experimental section for about 36 hours prior to paving This caused slight raveling of the wheel paths in the northbound lane with a maximum depth of slightly less than 1/2-inch. This indicates a surface coat of MS-4 emulsified asphalt may be needed if the completed section will be subjected to several days of traffic before paving.

Samples of field mixed aggregate were taken after the initial mixing and compacted into Marshall stability samples while the contractor was compacting the test section In addition, samples of the field mixed aggregate were taken after the test section was re-aerated to reduce its moisture content These samples were placed in plastic bags to minimize moisture loss and later compacted in the laboratory into Marshall stability samples

Laboratory sieve analysis tests show that loading, hauling, grading, and static rolling increased the modified subbase aggregate fines content about 2% The mixing operation increased the aggregate fines content about 2% Based on sieve analysis of Marshall sample remains, it is estimated that compaction increased the percent fines about 1 4%

The asphalt content was determined for several samples of field mixed aggregate The average asphalt content was 4 5%, identical to the specified value This asphalt content and the field moisture contents after the time of initial asphalt application (6.6%) and after re-aeration (4%) were used to prepare laboratory mixed Marshall stability samples

The Marshall stability and flow index of the field and laboratory mixed samples were very similar. However, the density of the field compacted samples were lower, probably because the field samples had a lower temperature at the time of compaction. The lower density did not seem to reduce the stability or flow index of the field compacted samples This shows that laboratory mixed samples can be used to predict the stability and flow index of field mixed samples.

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CHAPTER 6

CALCIUM CHLORIDE STABILIZED BASE

PRECONSTRUCTION LABORATORY TESTING

Nunan and Humphrey (1989) reviewed application rate recommendations from several suppliers. It was found that an application rate of 0 75 gal/yd^2 of 35% liquid calcium chloride solution was often recommended for stabilization applications Therefore, that rate was used for the experimental highway and preconstruction laboratory testing was unnecessary

CONSTRUCTION OF CALCIUM CHLORIDE STABILIZED SECTION

Calcium Chloride Mixing Operation

The contractor constructed the calcium chloride stabilized section on 12 September 1990. The weather was cloudy and breezy, with occasional periods of sun The contractor had previously placed the modified subbase aggregate in this section as described in Chapter 3 and compacted the aggregate with a static roller after placement

The contractor's initial construction task was to loosen up the aggregate The contractor performed this operation using the tractor and roto-tiller previously described for soil-cement mixing The contractor made approximately two passes of the roto-tiller The moisture content of the loosened soil measured with a nuclear density gauge before application of the calcium chloride was about 6 0% plus or minus 0 5% An average soil moisture content of about 7 0% plus or minus 0 5% was determined with a Coleman stove

The liquid calcium chloride supplier, W H Shurtleff Co , used a tractor trailer tanker equipped with a 12 foot wide spray bar at the end of the trailer This equipment is typically used to apply calcium chloride on gravel roads for dust control The contractor had difficulty achieving controlled application rates near the transition zones using this equipment The distribution control valve was located at the front end of the tank trailer. Therefore, after the valve was opened, the 3-inch distribution line located along the belly of the tank had to fill before flowing through the spray bar The initial application over the southbound lane began somewhat beyond the planned transition zone as a result Subsequent passes began as planned, but there was some delay between the beginning of distribution and forward movement of the tank truck After shutting the distribution control valve near the end of the section, all of the calcium chloride solution in the distribution line had to drain out Thus, the contractor applied a disproportionate amount of calcium chloride at the end of this experimental section More appropriate distribution equipment might have been equipped with a control valve at the spray bar, eliminating delay between actuating the valve and start and stop of flow

The supplier attempted to distribute the calcium chloride in three approximately equal applications at the rate of 0 25 gal/yd^2 The first application covered the entire 200 foot section To create the transition zone considering the difficulties experienced with distribution of calcium chloride as described above, the tank truck operator was directed to start and stop the second and third applications 12 and 25 feet from the ends of the section, respectively The contractor applied a total of 380 gallons over the section, about 8% more than the 350 6 gallons needed Most of the excess was released at the end of this section (approx STA 1036+20) due to the tanker control valve configuration

The contractor mixed the treated aggregate with three passes of the roto-tiller. It was observed that the mixed aggregate had a consistency similar to laboratory mixed calcium chloride treated samples through the full depth of loose stabilized aggregate Also noted during the mixing operation was that the last 20 feet of the section (STA. 1036+00 to 1036+20) contained 6-inch minus material The contractor apparently graded some of the standard subbase material from the succeeding control section into the calcium chloride section When mixing was complete, the contractor graded the aggregate with a Komatsu GD 605A motor grader.

Calcium Chloride Stabilized Base Compaction, Fine Grading Operations

The contractor compacted the stabilized aggregate with an Ingersoll-Rand Model SP 56 DD vibratory roller Project specifications required a minimum compaction level of 95% of maximum dry density as determined by an AASHTO T180 (ASTM D 1557) moisture density test performed on stabilized soil Moisture density test results were not available at the time of construction However, the MDOT field technicians were directed to use an estimated maximum dry density value of 134 4 pcf in their field density tests After construction of the experimental highway, a laboratory moisture density test was performed in conformance with AASHTO T180 The test resulted in a maximum dry density of 140 4 pcf at an optimum water content of 7 3% Based on a maximum dry density of 140 4 pcf, two of the four field density tests in the compacted aggregate exceeded the specified compaction level and two failed At least four passes of the roller were required to achieve the specified density but four passes was apparently inadequate in some areas All of the field in-place density and moisture content test results are presented in Table 6.1. After compaction, the contractor fine graded the stabilized aggregate with a Komatsu GD 605 A motor grader, then recompacted the graded surface.

Final Observations Before Paving

The calcium chloride stabilized section was inspected just before the contractor began paving on 19 September 1990 Traffic had been allowed to travel over the experimental section for about 36 hours after a 5-1/2 day curing period Some raveling was noted in the northbound wheel paths which had a maximum depth less than 1/2-inch There was a small amount of loose material at the surface as a result of raveling. This indicates that traffic on the unpaved

Table 6 1 Calcium chloride stabilized base moisture content and in-place density test results

STATION	NO. OF PASSES	SOIL MOISTURE %	PERCENT COMPACTION
1035+25 Lt Lane	1	6 5	85 2
1035+25 Lt Lane	2	6 3	87 5
1035+25 Lt Lane	3	6 5	88 3
1035+25 Lt Lane	4	6 7	93 3
1034+80 Rt Lane	--	6 2	96 4
1035+20 C/L	--	6.1	96 1
1035+80 Rt Lane	--	5.7	90.4
--		6.8*	
--		6.8*	
--		7.5*	

* These are aggregate water content test results of random samples before treatment These tests were performed with a Coleman stove

Note that approximately 6% moisture was measured in nuclear density gauge moisture content tests before application of calcium chloride treated section should be limited as much as practical Barely detectable deflection of the stabilized surface was observed as large, heavily loaded pulp trucks traveled over the section Chapter 3 discusses the paving operations and subsequent Road Rater pavement deflection measurements

Aggregate Sampling and Compacted CBR Samples

Modified subbase aggregate samples were collected from the calcium chloride stabilized section before the contractor distributed the calcium chloride The samples were used for post-construction laboratory grain size, pre-treatment (background) calcium chloride content and post-construction laboratory prepared CBR tests Samples of treated aggregate were also collected for calcium chloride content and laboratory water content tests

Field compacted CBR samples were made while the contractor graded and compacted the calcium chloride stabilized section The field mixed calcium chloride stabilized aggregate was sampled from STA 1035+00, Lt Lane; STA 1035+25, Centerline, STA 1035+50, Lt Lane, and STA 1035+75, Rt Lane, after the contractor completed the mixing operation For each station, a sample was compacted into 6-inch diameter molds for later testing in the laboratory

The CBR test samples were prepared in general conformance with ASTM D 1883, Bearing Ratio Of Laboratory-Compacted Soils (Appendix A) This standard requires that the samples be prepared and compacted in accordance with ASTM D 698 (AASHTO T 99), Moisture Density

Relations Of Soil Aggregate Mixtures Using 5 5 lb Rammer and 12 in Drop In the field, it was not possible to perform the specified oversize correction However, the field procedure was adapted by removing the particle sizes with a diameter greater than 3/4-inch determined by visual inspection The average dry unit weight of the field compacted CBR samples was 129 4 pcf

Plastic sealed with duct tape was used to cover the CBR molds after compaction, thus preventing evaporation The CBR samples were transported to the University of Maine the same day, protecting them against vibration by careful packing The covered CBR samples were allowed to cure at room temperature for 9 days before conducting the CBR tests

POST-CONSTRUCTION LABORATORY TESTING

Grain Size Analysis Tests

The modified subbase aggregate was sampled in the calcium chloride section before the contractor mixed it with calcium chloride. Grain size analysis tests were performed on this material and the results are presented in Appendix B, Figure 5 Similar to the other test sections, the sieve analysis test results show that degradation of the aggregate is a progressive process In this stabilized base section, loading, hauling, grading, and static rolling the modified subbase aggregate increased the fines content by 1 2% when compared to stockpile samples of the same aggregate (See Appendix B, Figure 1)

The treated aggregate was sampled after the contractor completed the mixing and compaction operations The results of sieve analysis tests performed on samples of this material are presented in Appendix B, Figures 6 and 7 Results indicate that the mixing and compaction operations increased the fines content an average of 2 5% in addition to the increase produced by aggregate placement

California Bearing Ratio (CBR) Tests

To replicate the calcium chloride field mixture for the laboratory prepared samples, it was necessary to first determine the average calcium chloride content of treated field trial aggregate samples Samples of treated aggregate from the top and bottom of the stabilized layer, remains of field compacted CBR samples, and samples of untreated modified subbase aggregate were used for calcium chloride content tests (Appendix A)

Table 6 2 presents the results of the calcium chloride content tests conducted on treated and untreated samples Note that samples from the top of the stabilized base layer have generally higher calcium chloride contents than samples from the bottom of the stabilized layer Also note a generally wide variability in calcium chloride content Mixing uniformity is less than ideal and indicates that visual verification of mix uniformity is inadequate The calcium chloride content is based on dry sample weight Thus, some of the variability may also be due to gravel content of the samples subjected to testing It is believed that more calcium chloride might

Table 6 2 Calcium chloride content test results

Sample No	Sample Location	Ca (mg/kg)	Cl (mg/kg)	Na (mg/kg)
1	STA 1034+50 L, Top	1600	2900	48
2	STA 1034+50 L, Bottom	1400	2500	39
3	STA 1035+00 R, Top	1600	2500	42
4	STA 1035+00 R, Bottom	1200	2000	34
5	STA 1035+50 L, Top	2800	5700	91
6	STA 1035+50 L, Bottom	1500	2500	43
7	Untreated Aggregate #1	180	6 8	6 5
8	Untreated Aggregate #2	150	9 5	7.3
9	Untreated Aggregate #3	190	23	9 3
10	Field CBR A, 1035+25 CL	2100	4700	69
11	Field CBR B, 1035+75 R	1700	3500	58
12	Field CBR C, 1035+00 R	1800	3500	56
13	Field CBR D, 1035+50 L	2000	4100	56

R = Right Lane

L = Left Lane

CL = Centerline

Note that the untreated aggregate samples were tested to determine average background element levels be found in a sandier sample because more particle surface area is available for the calcium chloride to adhere to

Calcium and chloride occur naturally in soil Therefore, the untreated samples were tested to determine background levels of calcium and chloride Based on the test results, two methods were used to calculate the amount of calcium chloride needed to prepare laboratory samples In one set of calculations, it was determined that 4 17 ml calcium chloride per pound of dry soil should be added to the subbase material In another set of calculations it was determined that 4 82 ml calcium chloride per pound of dry soil should be added to the subbase material The amount used to prepare calcium chloride stabilized aggregate samples for laboratory testing was 4 17 ml/lb Sample calculations for both methods are presented in Appendix F.

It is now believed that the 4 82 ml/lb value is more representative of the actual in-place calcium chloride content Thus, the laboratory samples needed approximately 14% more calcium chloride added to them before testing However, the test results showed that calcium chloride had a small affect on sample strength Therefore, it is concluded that increased calcium chloride content would not have significantly affected test results

The specifications required 4 69 ml calcium chloride (0 75 gal/yd^2) per pound of dry soil Therefore, the average calcium chloride content at the sample locations was about 3% more than the specifications required This is consistent with the observation that the subcontractor distributed more calcium chloride than was needed for the specified application rate

Grab samples of treated aggregate taken from the test section were subjected to laboratory water content tests The results of those tests are presented in Table 6 3 An average water content of 7 1% was determined for the treated aggregate It was calculated that 1 0% of the total moisture content was contributed by the calcium chloride solution Therefore, a soil moisture content of 6 1% was used to prepare the treated laboratory CBR test samples

Table 6 3 Laboratory water content test results from field samples of calcium chloride treated aggregate

Station		Water Content (%)
1034+50, Rt Lane		7 4
1034+50, Lt Lane		7 5
1035+00, Rt Lane		5 2
1035+00, Lt. Lane		7 9
1035+50, Rt Lane		6 4
1035+50, Lt. Lane		8 3
1035+75, Rt Lane		6 7
1035+75, Lt Lane		7 6
	Avg.	7.1

The average field moisture content of the aggregate in this experimental section prior to application of the calcium chloride solution was 7 0% (see Table 6 1) Therefore the untreated laboratory CBR samples were prepared using a 7 0% water content

Table 6 4 presents the CBR test results for the field compacted and post-construction laboratory compacted samples For comparison, laboratory tests included treated, untreated, as-compacted, and soaked CBR samples The effects of performing oversize correction during sample preparation as prescribed in the ASTM CBR test standard was investigated as discussed in Appendix A

The field compacted CBR samples were subjected to testing after curing for 9 days Two of the field compacted samples were soaked for 24 hours before testing Three of the untreated, laboratory generated, CBR samples were tested immediately after compaction and the remaining three after a 24-hour soaking period The treated, laboratory generated, CBR samples were tested after a 10-day curing period, including the 24 hour soaking period for those samples soaked before testing

A lower unit weight was observed in the field compacted samples This may be due to a loss of compaction energy since we placed the compaction molds on a heavy steel plate over a gravel surface to compact the field CBR samples Compared to field compaction, the concrete laboratory floor probably enhances compaction by forcing more of the compaction energy to be dissipated in the soil

Comparison of the CBR results shows that the field compacted

Table 6 4 CBR TEST RESULTS (%)

Station	Field Compacted Samples: As-Compacted	Field Compacted Samples: Soaked	Field Compacted Samples: Dry Unit Wt (pcf)[a]	Post Construction Samples, Treated: As-Compacted	Treated: Soaked	Treated: w[c]	Treated: Dry Unit Wt (pcf)	Untreated: As-Compacted	Untreated: Soaked	Untreated: Dry Unit Wt (pcf)[d]
1035+00 L		67	131 1							
1035+25 CL	53		127 2[b]							
1035+50 L		67	130 5							
1035+75 R	57		128 9							
Average	55	67	129 4							
				92[e]		7 1	134 6	72[e]		129 1
				77		6 4	131 0	61		134 6
				83		6 4	131 6	83		134 6
					80[e]	7 8	128 2		62[e]	129 2
					89	7 6	131 1		73	131 4
					72	7 0	130 1		75	130 4
Average				84	80	7 1	131 1	72	70	131 6

NOTES

a Dry unit weights calculated using oven-dry moisture contents of field samples

b Dry unit weight calculated using average oven-dry moisture content from STA 1035+00, L and R, and 1035+50, L and R, field samples

c Treated aggregate moisture content at time of laboratory preparation

d Samples prepared using 7 0% moisture based on field water content tests on untreated aggregate (see Table 6 1)

e These samples were prepared from 3/4-inch minus material only, no oversize correction

All other laboratory samples prepared using oversize correction samples had a lower CBR than the treated laboratory samples This may be due in part to the lower unit weight of the field compacted samples In addition, field mixing by the roto-tiller caused a 3 to 4 percent increase in fines It is unlikely that laboratory mixing and compaction produced a similar increase A higher fines content in the field samples would tend to cause a lower CBR Comparing the treated and untreated post construction samples, it is seen that the CBR for the treated samples is about 17% higher for as-compacted and 14% higher for soaked This is a much smaller improvement than found by Nunan and Humphrey (1989) for subbase aggregate from the Eagle Lake area which had an untreated CBR of about 30 It is possible that significant strength improvement by stabilization with calcium chloride may only be realized for soils with inherent low stability However, long term monitoring may show a benefit as a result of the different freeze-thaw characteristics CBR values were similar for both samples corrected for oversize aggregate per ASTM and samples using only 3/4-inch minus aggregate Thus the field procedure of using only 3/4-inch minus aggregate to prepare samples seems adequate Soaking had no apparent effect on the CBR values as evidenced by results which were both above and below the values for as-compacted samples

SUMMARY

The calcium chloride test section was constructed in general conformance with the specifications However, the amount of calcium chloride distributed by the contractor in this experimental section was about 8% more than specified (350 6 gal specified vs 380 gal distributed) due to the tanker control valve configuration This is consistent with calcium chloride content laboratory tests which indicated a calcium chloride content about 3% greater than specified 4 82 ml/lb vs 4 69 ml/lb) Most of the excess was applied near the end of the section at approximate STA 1036+00 to 1036+20 Visual observations of the treated aggregate indicated that the calcium chloride was well mixed into the aggregate However, the calcium chloride content tests indicated that about 1/3 more calcium chloride occurred in the top part of the stabilized layer, suggesting that more thorough mixing is needed Based on a post-construction compaction test, two of the four density tests in the compacted stabilized base met the specified compaction level

During construction of this experimental section, CBR samples were compacted with field mixed soil Treated and untreated aggregate samples were collected for laboratory grain size, calcium chloride content, water content, and laboratory compacted CBR tests

After a 5-1/2 day curing period, traffic was allowed over the calcium chloride stabilized base for about 36 hours Some raveling with a maximum depth less than 1/2-inch occurred in the northbound wheel paths Thus, traffic on unpaved treated sections should be limited Barely detectable deflections of the stabilized base surface were detected when large, heavily loaded pulp trucks traveled over the section The contractor paved over the calcium chloride section 7 days after it was constructed

Laboratory sieve analysis tests show that loading, hauling, grading, and static rolling increased the modified subbase aggregate fines content 1.2% Mixing and compaction of the treated aggregate increased the fines content an additional 2.5%

Laboratory CBR samples were prepared replicating field calcium chloride and soil moisture conditions, as well as untreated CBR test samples The untreated samples were immediately subjected to CBR tests or soaked for 24 hours and tested Treated samples cured for 9 or 10 days, including the 24 hour soak period for those samples soaked before testing

Field compacted CBR samples had lower CBR values than treated laboratory compacted CBR samples. Lower unit weights determined for field compacted samples may be due to compaction energy loss during the field compaction procedure which ultimately may have affected the CBR test results. Higher fines content as a result of the field mixing operation may also have contributed to the lower field CBR Comparison of laboratory compacted CBR test results shows that treated samples have CBR's 14% to 17% higher than untreated samples This improvement was less than found for aggregate from the Eagle Lake area which had a much lower untreated CBR It may be that calcium chloride stabilization gives a significant improvement only for soils with a low untreated CBR

Based on the CBR test results, calcium chloride did not significantly increase soil strength at the Van Buren test site Field and laboratory sample test results are nearly the same However, long term monitoring may show a benefit due to different freeze-thaw characteristics Soaking and oversize correction also had no significant effect on the CBR test results In view of the test results, more research is needed to compare the benefit of calcium chloride stabilization for aggregate with different untreated CBR's

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CHAPTER 7

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

Background

Aggregates used for road construction in Northern Maine often have poor strength characteristics The aggregate is mechanically broken down during construction processes, particularly by compaction and construction traffic Aggregate degradation increases the fines content of the base material immediately beneath the pavement. Thus, this layer is susceptible to frost action and has lower strength resulting in premature pavement failure in the form of rutting and cracking of the pavement surface.

Gravel stabilization is a potential method of improving the performance of this aggregate when used as a base material Previous research (Nunan and Humphrey, 1989) determined that soil-cement, asphalt, and calcium chloride stabilized subbase aggregate were potentially appropriate stabilization methods in terms of constructability, improved strength, and improved durability To further investigate the suitability of these three stabilization methods, MDOT sponsored this research and construction of a full-scale experimental highway test section as part of MDOT Project No 2586 This project comprises a 2 2 mile long total reconstruction of Route 1, beginning about two miles south of Van Buren, Maine

Current research facilitated evaluation of the constructability and short term performance of soil-cement, emulsified asphalt, and calcium chloride stabilization methods in a full-scale field trial The predictability of stabilization field performance from laboratory tests was also evaluated by comparing test results from field-generated and laboratory-generated samples The information gathered in this project will facilitate future performance monitoring of the above-described stabilization methods. Recommendations for long term performance monitoring are given in Appendix G.

Stabilization Test Sections

The 1020 foot long section of full-scale experimental highway is comprised of three stabilized and two control sub-sections, each 200 feet long. A 20 foot long untreated section was also constructed between the asphalt and calcium chloride stabilized sections Sub-grade and subbase preparation were identical for all the stabilized and control sections Vehicular and construction traffic traveled over the exposed subbase surface for several weeks Just prior to construction of the test sections, traffic was diverted to the shoulders beyond the limits of the stabilized and control section travel lanes The upper few inches of degraded subbase aggregate was then graded to the shoulders Grain size analysis tests of samples from the remaining subbase aggregate had fines contents ranging from

5 2% to 9 5%, indicating degradation of the aggregate and potential frost susceptibility

The stabilized sections and the modified subbase section were constructed between 10 and 12 September 1990 The weather was typically fair and warm, but the temperatures dipped into the mid-thirties one night The stabilized base course was constructed with modified subbase aggregate in the three stabilized base sections and the modified subbase control section Standard subbase aggregate was used to construct the standard subbase control section The modified subbase aggregate was similar to standard subbase aggregate (MDOT Specification 703.06b, Type D), but had a 2-inch maximum particle size to permit subsequent mixing operations The average fines content of stockpiled modified subbase aggregate was about 4 5%. It was determined that loading, hauling, and grading the modified subbase aggregate began the degradation process, increasing fines to between 4.7% and 6.4%. Mixing and compaction further increased the fines content Sieve analysis results show that fines contents ranged between 7.7% and 10.6% for mixed and compacted asphalt and calcium chloride stabilized aggregate

The soil-cement section was constructed using 6% cement based on preconstruction laboratory testing Aggregate moisture content was at the optimum when construction of this section began Bags of unopened Type I cement were spread at the spacing that would produce the desired cement content The spreading operation was begun by opening the bags and spreading with hand rakes The contractor then used a spring tooth harrow to evenly distribute the cement Subsequent laboratory tests indicate that the contractor achieved a 6 4% cement mixture Research required performance of field water content tests, gathering untreated aggregate samples for grain size analysis and laboratory mixed soil cement cylinders, and compaction of soil-cement cylinders using field mixed soil-cement for later laboratory testing

A 7-day compressive strength between 300 and 800 psi will provide a durable soil cement base (PCA, 1971). All of the field compacted cylinders, including compression, wet-dry, and freeze-thaw test cylinders, exhibited 7-day compressive strengths within this range Laboratory prepared samples had compressive strengths 200 to 400 psi greater than the field compacted samples The maximum allowable soil-cement loss after 12 cycles of wet-dry or freeze-thaw testing is 14 percent (PCA 1971) All of the freeze-thaw and wet-dry test results for tests on field compacted and laboratory compacted soil-cement cylinders met the PCA criteria.

The asphalt stabilized section was constructed using 4 5% MS-4 emulsified asphalt as determined by preconstruction laboratory testing It was estimated that a moisture content of less than 5% would produce a stable mix with this asphalt content Since the initial moisture content of the aggregate was about 8% when it came from the borrow pit, the contractor's first task was to reduce the water content This was done by tilling the exposed aggregate with the rototiller After several hours, the contractor was able to reduce the water content to 6 6% The contractor was then directed to distribute and mix the emulsified asphalt although the aggregate moisture content was still higher than desired with the understanding that the contractor would re-aerate the stabilized aggregate several days later As expected the mix was unstable with a moisture content of 6.6%

Re-aeration reduced the water content to 4 0% which produced a stable mixture in general conformance with the project specifications. Asphalt extraction tests show that the contractor achieved the specified target of 4.5% asphalt in the stabilized mixture Research required performance of field water content tests, gathering treated and untreated aggregate samples for grain size, water content, and laboratory mixed Marshall tests, and compaction of field mixed Marshall samples for later laboratory testing. Random samples of the re-aerated aggregate were also collected to compact Marshall samples comprising the lower moisture content

A minimum stability of 500 lbs. is specified as the Marshall design criteria for medium traffic (The Asphalt Institute, 1974) All of the field compacted and post-construction laboratory compacted Marshall samples met that criteria. The average field compacted Marshall stability values were 683 lbs and 1440 lbs for 6 6% and 4 0% moisture, respectively The average laboratory compacted Marshall stability values were 751 lbs and 1433 lbs for 6 6% and 4 0% moisture, respectively Marshall stability results varied less than 10% between field and laboratory samples The flow index of the field and laboratory mixed samples were also similar

The calcium chloride section was constructed using an application rate of 0 75 gal/yd^2 of 35% calcium chloride solution as recommended by calcium chloride dealers Aggregate moisture was within the range required for compaction when construction of this section began Equipment difficulties resulted in distribution of varying amounts of calcium chloride, in particular more calcium chloride was applied near the end of the section The contractor performed the mixing and compaction operations in general conformance with the project specifications Calcium chloride content tests show varying calcium chloride content, and the average was about 3% above the specified values. Also, more calcium chloride was present at the top of the stabilized layer than at the bottom, indicating that mixing was inadequate. Research required performance of field water content tests, gathering treated and untreated aggregate samples for grain size, water content and laboratory mixed CBR tests, and compaction of field mixed CBR samples for later laboratory testing

Laboratory CBR test results for untreated modified subbase aggregate averaged about 71 Laboratory CBR test results for treated aggregate averaged about 82. Test results from field compacted CBR samples averaged about 61. Stability improvement for the Van Buren project is less than found for aggregate from the Eagle Lake area which had a much lower untreated CBR

Road Rater Pavement Deflection Measurements

Pavement deflections were measured on 28 September 1990, 21 May 1991, and 6 August 1991. Results are presented in Table 3 2 Although no deflection criteria exist for interpreting the results, a deflection exceeding 5 mils is considered undesirable (Nunan and Humphrey, 1989) For all three dates, the sensor readings were below this value The sensor 1 readings are more indicative of near surface variation in strength and will be used in the following comparison Soil cement consistently showed the smallest sensor 1 deflection On 28 September 1990, the remaining four sections showed very similar deflections However, on 21 May 1991 and 6 August 1991, the asphalt stabilized section showed the second lowest deflection, while the deflections for modified subbase, calcium chloride stabilized, and standard subbase were somewhat larger This suggests that soil cement provides the largest structural benefit, with asphalt providing a lesser benefit. Calcium chloride provided no discernible increase in strength compared to untreated sections Note that the total pavement depth was 3, 4-3/4, and 6 inches for the 28 September, 21 May, and 6 August Road Rater events, respectively.

Surveyed Cross-Sections

MDOT performed cross-section surveys at planned profile levels, from subgrade to final paving as construction progressed The surveyed cross-sections will permit the future evaluation of the magnitude and source of rutting Knowing the source of the rutting will assist in formulating appropriate modifications to the design

CONCLUSIONS

Based on the work presented in this report, the following conclusions are made

Aggregate Degradation

1 Grain size analysis showed that the standard subbase aggregate beneath the stabilized layer has excess fines as a result of compaction and construction traffic and may therefore be susceptible to frost action

2. Grain size analysis tests showed that degradation of the modified subbase aggregate is a progressive process Loading, hauling, and grading the aggregate resulted in total fines contents ranging between 4.7% and 6.4% while stockpiled aggregate contained an average 4.5% fines Mixing and compaction further increased fines in the stabilized sections to total fines contents ranging between 7 7% and 10.6%.

Soil-Cement Stabilized Base

1 The strength, freeze-thaw, and wet-dry test results were significantly less for the field mixed samples than the lab mixed samples This is most likely due to weak bond strength resulting from a fine silt coating on the aggregate in the field mixed samples The lab procedure required that the plus No 4 aggregate be saturated before sample preparation which washed away the silt coating Other factors contributing to lower

values for the field samples include a lower unit weight, curing temperature and humidity for the field samples, delay between mixing and compaction in the field, and field mixing may have resulted in a less uniform mix The difference between the freeze-thaw and wet-dry results for the field mixed and laboratory mixed samples was so large that it was concluded that for this aggregate type, laboratory mixed samples give no useful indication of field durability

2 The field procedure where visually determined 3/4-inch plus aggregate is removed before compacting soil-cement into the mold gives nearly the same strength and durability results versus the AASHTO lab procedure which requires an oversize correction Thus, the field procedure appears adequate

3 The fine grading operation needs careful planning to control grade, prevent undercutting some areas, and keeping excess stabilized aggregate within the outer limit of the stabilized width During the field trial, it was observed that some of the material brought back from the windrow to fill undercut areas had inadequate cement content

4 A durable soil-cement mixture was achieved at the Van Buren field trial project site All of the soil-cement compression test results and the percent soil-cement loss from freeze-thaw and wet-dry tests on the field compacted cylinders fell within the PCA criteria

Asphalt Stabilized Base

1 For the aggregate tested, laboratory mixed Marshall samples give about the same results as field mixed samples Therefore, laboratory mixed samples can be used to predict the behavior of field mixed samples

2 The Marshall stability test procedure requires that particles larger than 3/4-inch be screened out The field procedure where visually determined 3/4-inch plus aggregate is removed before compacting Marshall samples gives nearly the same strength and durability test results as screening out the plus 3/4-inch particles. Thus, the field procedure appears adequate

3 A surface coat of MS-4 asphalt is needed to protect the cured asphalt stabilized base if the completed section will be subjected to traffic for more than a few days before paving.

4 The stability of the compacted asphalt stabilized base is very sensitive to the initial aggregate moisture content For an asphalt content of 4 5%, an initial aggregate moisture content of less than about 5% is needed to produce a stable mix.

5 A durable asphalt stabilized base was achieved at the Van Buren field trial site. Stability results for all of the field compacted samples met the Asphalt Institute design criteria of 500

lbs for medium traffic highways However, the difficulty of reducing the initial aggregate moisture content to an acceptable level makes this method impractical for the climatic conditions encountered in northern Maine if in-place mixing methods are used

Calcium Chloride Stabilized Base

1 Laboratory compacted CBR test results showed only 14% to 17% higher CBR values for treated/soaked and treated/as-compacted samples as compared to untreated samples. The results of Nunan and Humphrey (1989) suggest that higher strength gains may be possible for aggregate with a lower untreated strength, such as that found in the Eagle Lake area Soaking has no significant affect Additional study is needed to investigate the relation between CBR values and field performance of calcium chloride treated base

2. More thorough mixing of the calcium chloride stabilized base is needed. Visual observations of mix uniformity are inadequate Calcium chloride content tests showed about 1/3 more calcium chloride in the top versus the bottom of the stabilized layer in spite of uniform mixture appearance

3 Traffic over unpaved treated sections should be limited to prevent raveling

4 The field procedure where visually determined 3/4-inch plus ag-

gregate is removed before compacting calcium chloride stabilized aggregate into CBR molds gives nearly the same strength results as the ASTM lab procedure which requires an oversize correction. Thus, the field procedure appears adequate

5 Long term monitoring may show a benefit due to different freeze-thaw characteristics

RESEARCH RECOMMENDATIONS

General

1 Long-term monitoring should be carried out as described in Appendix G

Soil-Cement

1 The construction of the soil-cement section for this field trial required separate travel lanes to maintain traffic while the soil-cement cured Therefore, a worthwhile consideration is the effect of loading on soil-cement stabilized aggregate after various curing periods, such as, 1 day, 2 days, 3 days, etc Obviously, the shortest time that produces reasonable stabilization will help the contractor execute construction at a reasonable cost.

2 Compressive strength and durability of field compacted soil-cement samples was lower than for laboratory compacted samples The following investigations may help define the reasons for

the strength differences and develop laboratory procedures which produce results that are more representative of field behavior

- investigate the variation in bond strength based on clean versus silt coated aggregate,
- investigate the effect of varied time delay after mixing and before compaction,
- investigate the effects of various curing methods, for example, varying temperature and humidity, replicating field conditions,
- investigate the relationship between compactive effort, unit weight and compressive strength

Asphalt

1. High initial aggregate water content limits the usefulness of this stabilization method Equipment and mixing methods other than mixing in place with a roto-tiller should be investigated to determine whether or not other cost effective processes are in use for preparing asphalt stabilized base materials

Calcium Chloride

1 The relationship between inherent stability and the amount of potential stability improvement by stabilization with calcium chloride should be investigated Nunan and Humphrey (1989) found significant stability improvement in soils with low CBR values Current research indicates that aggregate with relatively high CBR values do not gain significant stability by

calcium chloride stabilization Other northern Maine aggregates should be treated and tested to determine if in fact there is a correlation between inherent strength and potential strength improvement

A trial calcium chloride section should be constructed on a road whose subbase aggregate has an untreated CBR of less than 50 It is likely that aggregate from the Eagle Lake area will meet this criterion. This test section would show the effect of calcium chloride on aggregate whose strength is significantly less than at the Van Buren site In addition, the effect of varying the calcium chloride application rate should be investigated.

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APPENDICES

APPENDIX A

FIELD AND LABORATORY TEST PROCEDURES

Field And Laboratory Test Procedures

Introduction

In general, field and laboratory test procedures conformed to the corresponding AASHTO and ASTM Standard Designation for specific material properties tests The authors prepared samples in the field using equipment normally used in the laboratory for a given test For example, we prepared Marshall stability samples using a hot water bath for the molds and hammer, mold retainer/compaction stand, and the Marshall hammer Thus, we attempted to replicate laboratory procedures in the field as much as possible

General descriptions of the test procedures and the AASHTO and ASTM Standard Designation used as a guide are presented in the following text. The test results are either tabulated in the body of the report, or are placed in appendices as appropriate.

Water Content

The moisture content is the ratio expressed as a percentage of weight of water in a given mass of soil to the weight of the solid particles. In the laboratory, we conducted this test in accordance with ASTM Designation D 2216, Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil - Aggregate Mixtures. However, field moisture contents were determined by nuclear density gauge and/or using a Coleman stove to drive off sample moisture

We tried to correlate Speedy moisture results with oven-dry samples It was initially thought that the Speedy moisture would help the contractor during construction of the soil-cement section by producing rapid moisture content results We found, however, that the Speedy moisture results were inconsistent The experimental nature of the project required more accurate and consistent results as were achieved by the nuclear gauge and Coleman stove

Grain Size Distribution

Samples selected for grain size analysis were quartered, weighed, washed over a No 200 mesh sieve, dried, and re-weighed We then passed the dried remaining sample over 3-inch through No 200 sieves to determine the grain size distribution of the sample To determine the weight of material in the silt and clay size range, we added the soil removed by washing and the 200 minus material collected in the sieving pan This test is similar to that described by ASTM Designation D 422, Particle-Size Analysis of Soils

Field Soil Density Tests

After compaction, stabilized and untreated soil densities were measured in the field with a nuclear density gauge The procedure used conforms to ASTM D 2922, Density Of Soil And Soil-Aggregate In Place By Nuclear Methods

Moisture-Density Tests

Moisture density tests were performed on soil-cement samples in accordance with ASTM D 558, Method B, Moisture-Density Relations Of Soil-Cement Mixtures. All of the soil was passed through a 3-inch, 3/4-inch, and No. 4 sieves. Material retained on the 3-inch sieve is discarded. Material retained on the No 4 and passing the 3/4-inch sieve is soaked. An oversize correction is performed by replacing material passing the 3-inch sieve and retained on the 3/4-inch sieve with an equal weight of material which passes the 3/4-inch sieve and is retained on the No. 4 sieve The 3/4-inch to No 4 material is soaked and surface dried before combining it into the sample

Cement is combined in the desired proportion to dry weight of soil and initially mixed with the material passing the No 4 Then the material passing the 3/4-inch sieve, but retained on the No 4, is combined. We developed a moisture-density curve by compacting the soil into a 4-inch mold at various water contents using a 5 5 lb hammer, 3 layers, and 25 blows per layer. The soils' maximum dry density is taken from the high point of the density vs. moisture content curve.

We also conducted a moisture-density test on calcium chloride stabilized base materials and untreated modified subbase aggregate. MDOT performed a moisture-density test on the standard subbase aggregate. The moisture-density tests also required an oversize correction and incorporates a 6-inch mold in place of the 4-inch mold. For the calcium chloride stabilized aggregate, the soil was prepared as normally done except that calcium chloride was added to the soil in the same proportion as was added to the soil in the field trial These tests were performed in general conformance with AASHTO T 180 (ASTM Designation D 1557, Method D, Moisture-Density Relations of Soils and Soil Aggregate Mixtures Using 10 lb Rammer and 18 in Drop) The maximum dry density of the stabilized soil is determined as described above

Unconfined Compression Test

Compressive strengths of molded soil-cement cylinders were determined by unconfined compression tests in accordance with ASTM D 1633, Compressive Strength Of Molded Soil-Cement Cylinders For the laboratory prepared samples, the soil-cement was mixed and compacted at the desired moisture and cement contents as described in the soil-cement moisture density test above and cured in a humid room for 7 or 28 days prior to the compression tests Freeze-thaw and wet-dry soil-cement cylinders were tested for residual compressive strength after 37 days

Soil-cement was mixed in-place for the field trial with a tractor and large roto-tiller driven by a power take off Immediately after mixing, bucket samples of the mixed material were taken and particles larger than 3/4-inch (estimated by visual inspection) were removed before compaction of the specimens Field compacted soil-cement cylinders were allowed to cure at outdoor ambient temperatures for several days to replicate field conditions of the soil-cement stabilized base materials The field compacted cylinders were then brought to the laboratory where they were placed in the humid room to complete the curing period. Because it was not possible to perform oversize corrections on the field generated samples, the post construction laboratory mixed tests included two iterations. One series of tests were conducted in the normal mode incorporating an oversize correction Another series of tests were conducted on samples prepared in a manner similar to the field conditions by using only the material that passed a 3/4-inch sieve.

Before testing, the cylinders were capped with a sulfur capping compound and soaked for 4 hours. The cylinders were placed on the Forney compression testing machine and a load was applied at a constant rate of 20 psi/sec plus or minus 10 psi. The maximum load at failure was recorded. The compressive strength of the sample is calculated by dividing the maximum load by the cross-sectional area.

Freeze-Thaw Test

The freeze-thaw durability of soil-cement mixtures was tested in general conformance with ASTM D 560, Freezing-And-Thawing Tests Of Compacted Soil-Cement Mixtures. For the laboratory prepared samples, the soil cement was mixed at the desired moisture and cement contents as described in the soil-cement moisture density test above and cured for 7 days The field mixed samples and post construction samples were prepared as described for the compression tests.

The samples were then subjected to 12 cycles of alternate 24 hour freezing and thawing periods The samples were placed in a freezer no warmer than -10 degrees Farenheit. During the thaw period, samples were placed in a humid room with a temperature and humidity of about 70 degrees Farenheit and 100%, respectively The samples were stroked a prescribed number of times with a wire scratch brush at the end of the thaw period After 12 cycles, the samples were dried and weighed The percent soil-cement loss is essentially the difference between initial and final dry sample weights

Wet-Dry Test

The wet-dry durability of soil-cement mixtures was tested in general conformance with ASTM D 559, Wetting-And-Drying Tests Of Compacted Soil-Cement Mixtures For the laboratory prepared samples, the soil-cement was mixed at the desired moisture and cement contents as described in the soil-cement moisture density test above and cured for 7 days The field mixed samples and post construction samples were prepared as described for the compression tests.

The samples were then subjected to 12 cycles of alternate wetting and drying periods. The samples were submerged in potable water for 5 hours during the wetting period and placed in a 160 degree Farenheit oven for 42 hours during the drying period. The samples were stroked a prescribed number of times with a wire scratch brush at the end of the drying period After 12 cycles, the samples were dried and weighed The percent soil-cement loss is essentially the difference between initial and final dry sample weights

Cement Content Test

Field compacted soil-cement cylinders were subjected to cement content tests after compression, freeze-thaw, and wet-dry tests were completed. The cement content was determined in accordance with ASTM D 806, Cement Content Of Soil-Cement Mixtures.

The samples were dried to remove free moisture and pulverized with a steel compaction test rammer so that everything passed a No. 40 sieve. It is important to note that all of the dry sample, with weight as prescribed in the above standard, must be pulverized to conduct this test correctly. A first attempt by the authors was unsuccessful because the sample was only partially pulverized, leaving the aggregate behind and resulting in erroneous high cement contents. After pulverizing, chemical analysis is performed to determine the cement content based on calcium oxide content. The percent cement in the soil-cement samples were determined from a comparison of the amount of calcium oxide in the untreated modified subbase aggregate to the calcium oxide in the soil-cement samples. The samples were prepared by the authors and the testing was carried out by MDOT, Technical Services Division

Marshall Stability Test

Asphalt stabilized soil samples were prepared and tested in general accordance with the Modified Marshall Test procedure except as noted below. For the laboratory prepared samples, the MS-4 emulsified asphalt was heated to approximately 150 degrees Farenheit The hot asphalt was added to room temperature soil to replicate field conditions where the subbase aggregate will be at ambient temperature before mixing In contrast to our test procedure, the Modified Marshall Test procedure requires heated soil be mixed with heated asphalt

The subbase aggregate moisture content was varied to evaluate the effect various water contents had on stability The soil and asphalt were blended into a uniform mixture and placed in a 4-inch diameter by 2 5-inch high Marshall mold fitted with a 2-inch high collar The sample was compacted from the top and bottom with 50 blows of a Marshall hammer The samples were then extruded from the mold and air dried at room temperature

All the laboratory equipment necessary to prepare Marshall samples was brought to the field trial site, including a portable generator to supply power to the water bath Field mixed samples were prepared from bucket samples taken from the mixed in place aggregate The field compacted samples were allowed to air dry for several days at outdoor ambient temperatures and for several days at room temperature in the laboratory.

After a cure period of 8 to 10 days, samples were placed in the Marshall test frame and loaded at a constant deformation rate of 2 inches per minute. The samples were not heated before testing The maximum load and the deformation at the instant the maximum load was reached were recorded. The maximum load was used to determine the Marshall stability number and the deformation reading was converted to the flow index.

Marshall stability sample unit weights were determined by applying the Archimedes principle. The Marshall samples were soaked for 30 to 60 minutes before testing. The saturated surface dry and submerged sample weights were recorded. The sample volume was calculated by subtracting the submerged sample weight from the saturated surface dry sample weight and dividing the result by the unit weight of water. The submerged unit weight is then calculated by dividing the submerged sample weight by the volume calculated above. Finally, the submerged unit weight is added to the unit weight of water and the sum is divided by 1 plus the water content of the dried sample remains, resulting in the dry unit weight of the sample mixture.

Asphalt Extraction Test

MDOT determined the asphalt content of selected field generated Marshall samples by conducting asphalt extraction tests The tests were conducted in general accordance with ASTM D 2172, Quantitative Extraction of Bitumen From Bituminous Paving Mixtures. Typically, the samples are washed in a bath of asphalt solvent The solvent is then drawn off and filtered to collect the asphalt A grain size analysis is then performed on the remaining sample

California Bearing Ratio (CBR) Test

Strength improvement in the calcium chloride stabilized soil was measured by the CBR test The tests were performed in accordance with ASTM D 1883, Bearing Ratio of Laboratory-Compacted Soils Calcium chloride treated and untreated subbase aggregate samples were compacted in CBR molds according to ASTM D 698, Method D, Moisture-Density Relations of Soils and Soil Aggregate Mixtures Using 5 5 lb Rammer and 12 in Drop ASTM 698 was used to compare CBR values derived in the Nunan and Humphrey (1989) investigations

CBR tests were performed on untreated samples immediately Treated samples were cured for 9 or 10 days before testing 15 lb surcharge weights were placed over the soil in the mold prior to testing The surcharge weight is approximately equal to the base material and pavement which will exist above the soil in the field The loading frame penetration piston is seated with a load no greater than 10 lbs. The sample is then loaded so that the rate of penetration is approximately 0 05 inch per minute The load is recorded at predetermined penetration depths

The bearing ratios are determined by using the corrected load values taken from the load/penetration curve for 0.1 inch and 0.2 inch penetrations and dividing them by the standard loads of 1000 psi and 1500 psi, respectively. The bearing ratio reported for the soil is normally the one at 0.1 inch penetration However, when the ratio at 0.2 inch penetration is greater, and check tests produce the same result, the bearing ratio at 0 2 inch penetration is reported.

Because it was not possible to perform oversize corrections on the field generated samples as required by ASTM D 698, the post construction laboratory testing of calcium chloride stabilized soil included two iterations One series of tests were conducted in the normal mode incorporating an oversize correction. Another series of tests were conducted on samples prepared in a manner similar to the field conditions by using only the material that passed a 3/4-inch sieve

Calcium Chloride Content Test

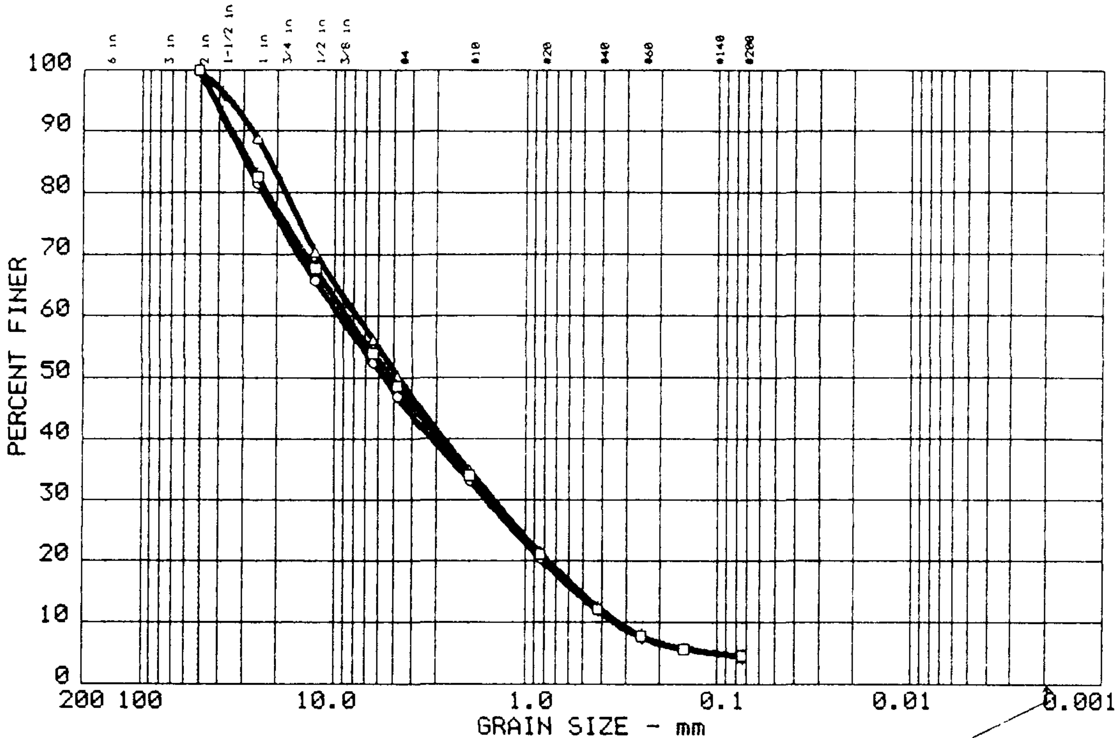
Following the field trial, it was necessary to test the calcium chloride treated soil to determine the actual chloride content This information would allow the construction of similar samples in the laboratory for comparative testing. The procedure used to determine the chloride content was ASTM D 1411, Water-Soluble Chlorides Present As Admixes In Graded Aggregate Road Mixes

We collected treated and untreated soil samples during construction of the calcium chloride stabilized highway section These samples were taken to the Dept of Plant and Soil Sciences at the University of Maine where the actual testing was performed

APPENDIX B

GRAIN SIZE DISTRIBUTION TEST RESULTS

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	53.1	42.4	4.5	
△	0.0	49.7	45.8	4.5	
□	0.0	51.3	44.1	4.6	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			29.04	9.40	5.59	1.613	0.5464	0.3408	0.81	27.6
△			21.63	7.66	4.67	1.477	0.5182	0.3270	0.87	23.4
□			28.15	8.60	5.12	1.529	0.5303	0.3308	0.82	26.0

MATERIAL DESCRIPTION	USCS	AASHTO
○ Mod. Subbase Mat'l. Stockpiled For Field Trial	GP	
△ Mod. Subbase Mat'l. Stockpiled For Field Trial	GP	
□ Mod. Subbase Mat'l. Stockpiled For Field Trial	GP	

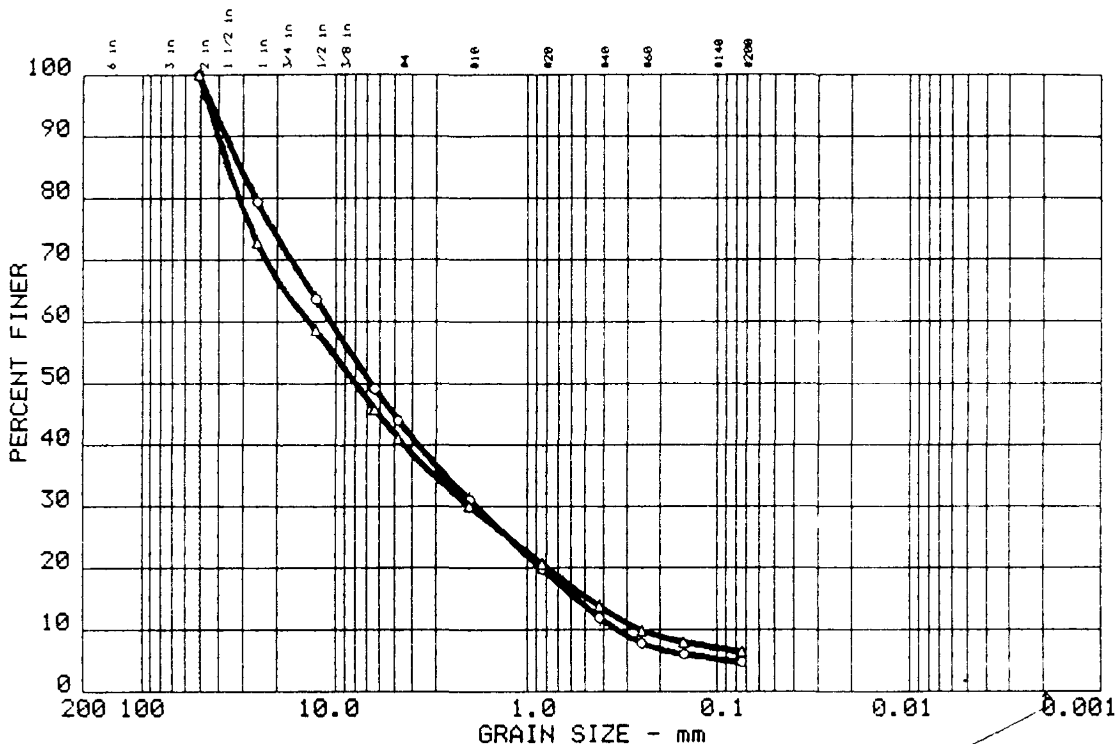
Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: Random Sample, Mod. Subbase Stockpile
△ Location: Random Sample, Mod. Subbase Stockpile
□ Location: Random Sample, Mod. Subbase Stockpile
Date: 8-21-90

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT

Figure No. 1

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	55.9	39.3	4.8	
△	0.0	58.8	34.8	6.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			31.15	10.68	6.58	1.856	0.5540	0.3377	0.95	31.6
△			35.85	13.79	7.93	1.970	0.4781	0.2538	1.11	54.3

MATERIAL DESCRIPTION	USCS	AASHTO
○ Mod. Subbase Material Before Adding Cement	GP	
△ Mod. Subbase Material Before Adding Asphalt	GW-GM	

Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: Random Sample, Soil Cement Test Sect.
△ Location: Random Sample, Asphalt-Soil Test Sect

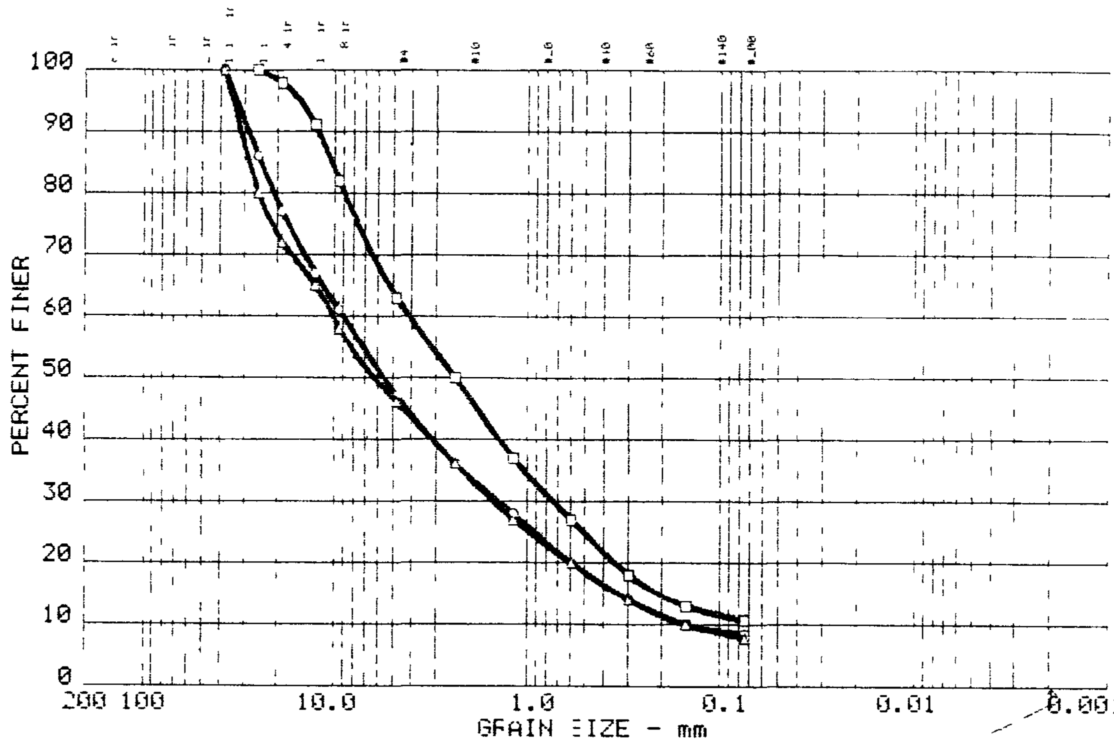
Date: 9-15-90

GRAIN SIZE DISTRIBUTION TEST REPORT

Remarks:

Figure No. 2

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	53.0	38.6	8.4	
△	0.0	54.0	38.2	7.8	
□	0.0	37.0	52.4	10.6	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			24.55	9.02	5.56	1.413	0.3311	0.1475	1.50	61.0
△			26.51	10.32	6.22	1.526	0.3300	0.1474	1.53	70.0
□			10.42	4.10	2.36	0.729	0.2080			

MATERIAL DESCRIPTION	USCS	AASHTO
○ Asphalt-Soil After Mixing	GW-GM	
△ Asphalt-Soil After Mixing	GW-GM	
□ Asphalt-Soil After Mixing **(Marshall Spl Remains)**	SP-SM	

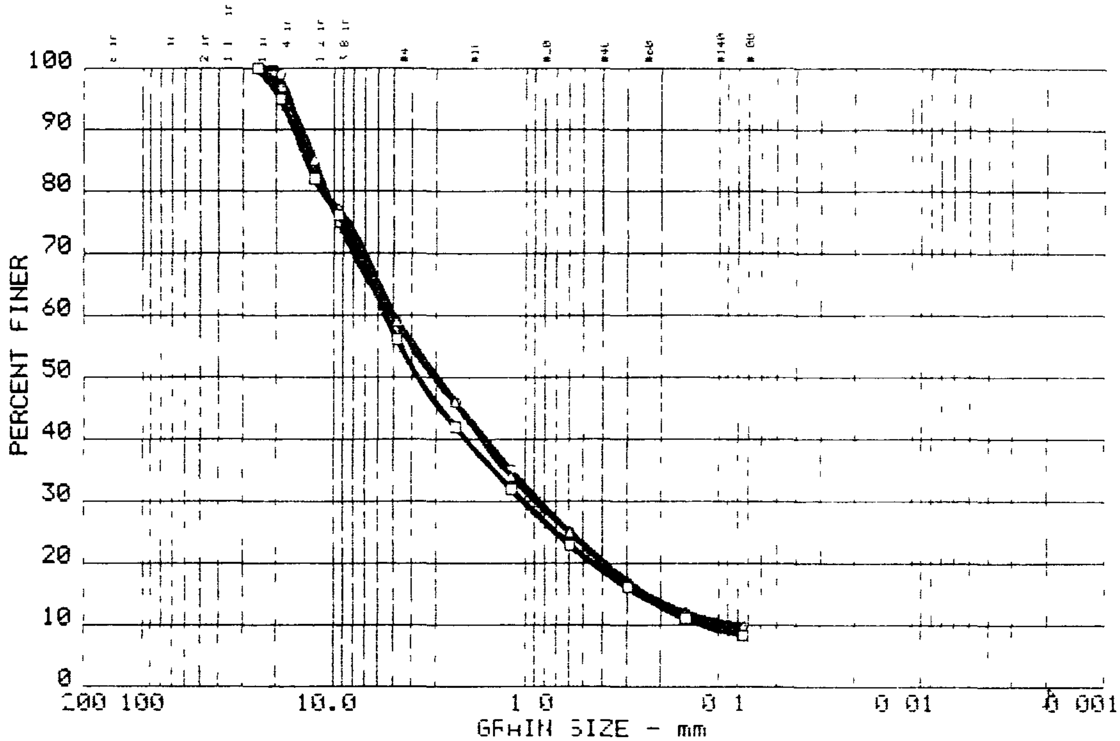
Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: Random Sample, Asphalt-Soil Test Sect
△ Location: Random Sample, Asphalt-Soil Test Sect
□ Location: STA. 1032+25, Lt Lane
Date: 9-28-90

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT

Figure No. **3**

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	41.0	50.0	9.0	
△	0.0	41.0	49.2	9.8	
□	0.0	44.0	47.7	8.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			13.72	4.93	3.04	0.946	0.2253	0.0939	1.55	52.5
△			12.69	4.99	2.94	0.888	0.2336	0.0801	1.97	62.4
□			13.96	5.43	3.72	1.023	0.2630	0.1175	1.64	46.2

MATERIAL DESCRIPTION	USCS	AASHTO
○ Asphalt-Soil After Mixing **(Marshall Spl Remains)**	SW-SM	
△ Asphalt-Soil After Mixing **(Marshall Spl Remains)**	SW-SM	
□ Asphalt-Soil After Mixing **(Marshall Spl Remains)**	SW-SM	

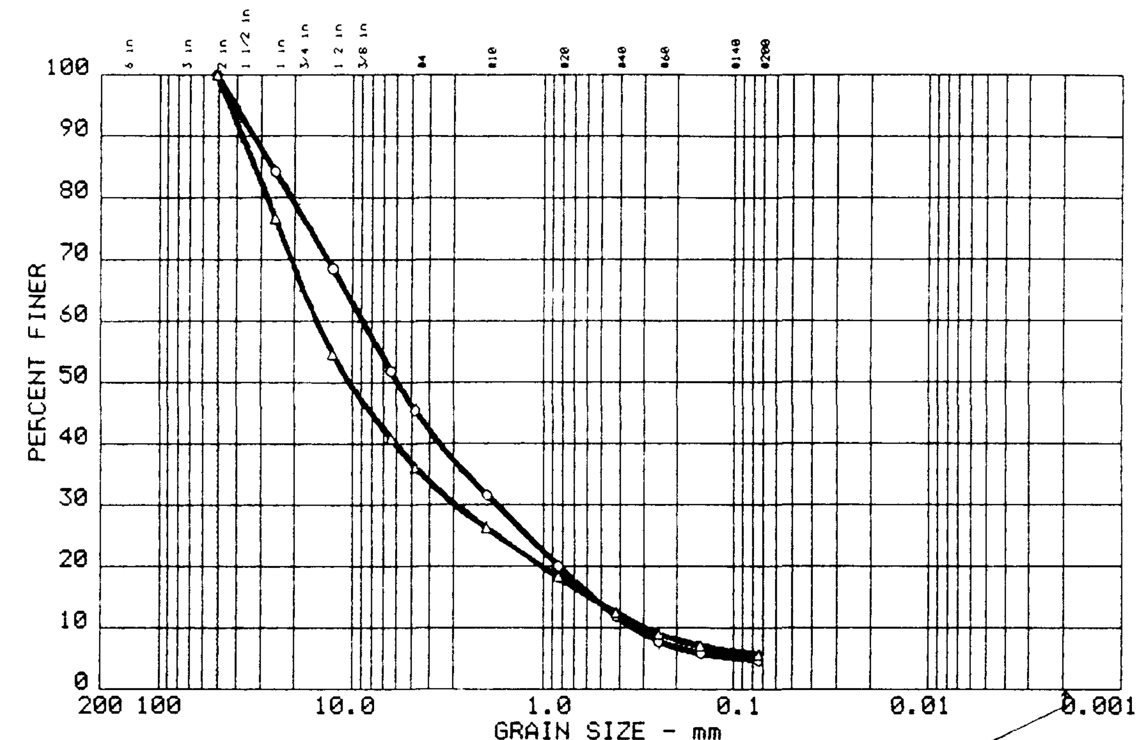
Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: STA. 1032+75, Rt. Lane
△ Location: STA. 1033+25, Lt. Lane
□ Location: STA. 1033+75, Rt. Lane
Date: 9-28-90

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT

Figure No. **4**

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	54.6	40.7	4.7	
△	0.0	63.8	30.5	5.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			26.27	8.89	5.87	1.772	0.5540	0.3377	1.05	26.3
△			32.36	15.31	10.35	2.917	0.5689	0.2951	1.88	51.9

MATERIAL DESCRIPTION	USCS	AASHTO
○ Modified Subbase Material Used In Control Sect.	GW	
△ Mod. Subbase Material Before Adding CACL2	GW-GM	

Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: Random Sample, Mod. Subbase Cont. Sec
△ Location: Random Sample, CACL2 Test Section

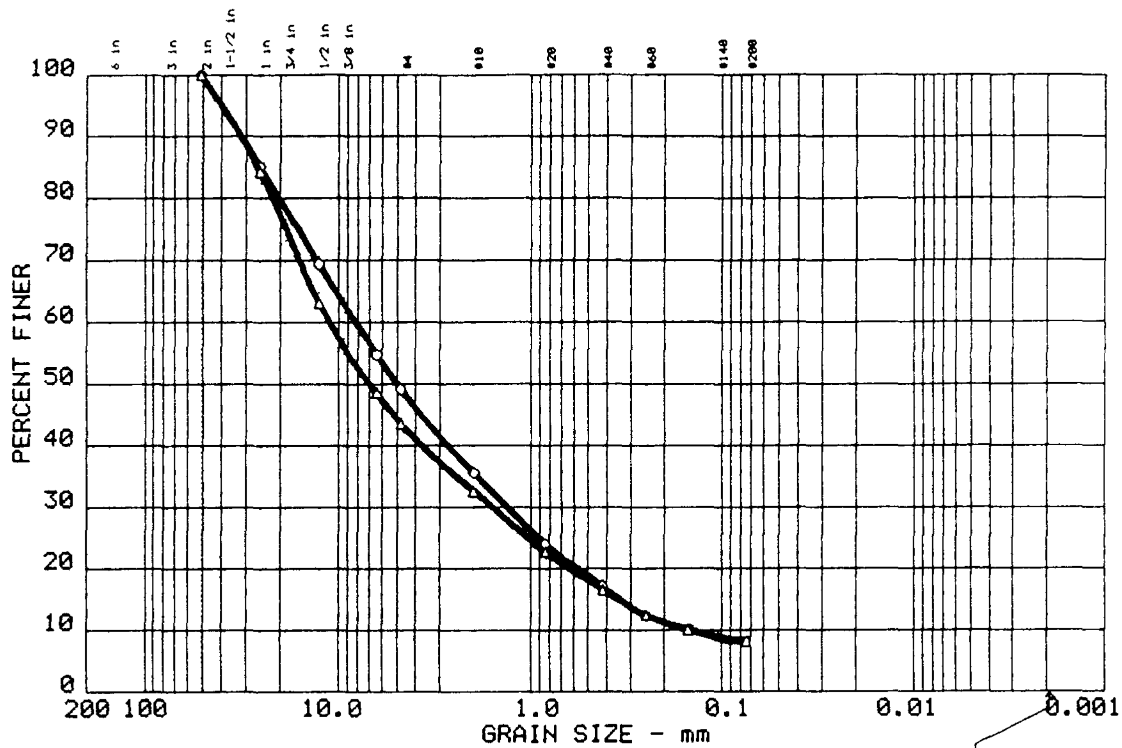
Date: 9-15-90

GRAIN SIZE DISTRIBUTION TEST REPORT

Remarks:

Figure No. 5

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	50.8	40.9	8.3	
△	0.0	56.4	35.5	8.1	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			25.38	8.12	4.95	1.347	0.3385	0.1427	1.57	56.9
△			26.00	11.22	6.84	1.585	0.3467	0.1479	1.51	75.9

MATERIAL DESCRIPTION	USCS	AASHTO
○ $CACL_2$ - Soil Mix After Compaction	GW-GM	
△ $CACL_2$ - Soil Mix After Compaction	GW-GM	

Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: $CACL_2$ Test Section, STA 1034+50L
△ Location: $CACL_2$ Test Section, STA 1035+00R

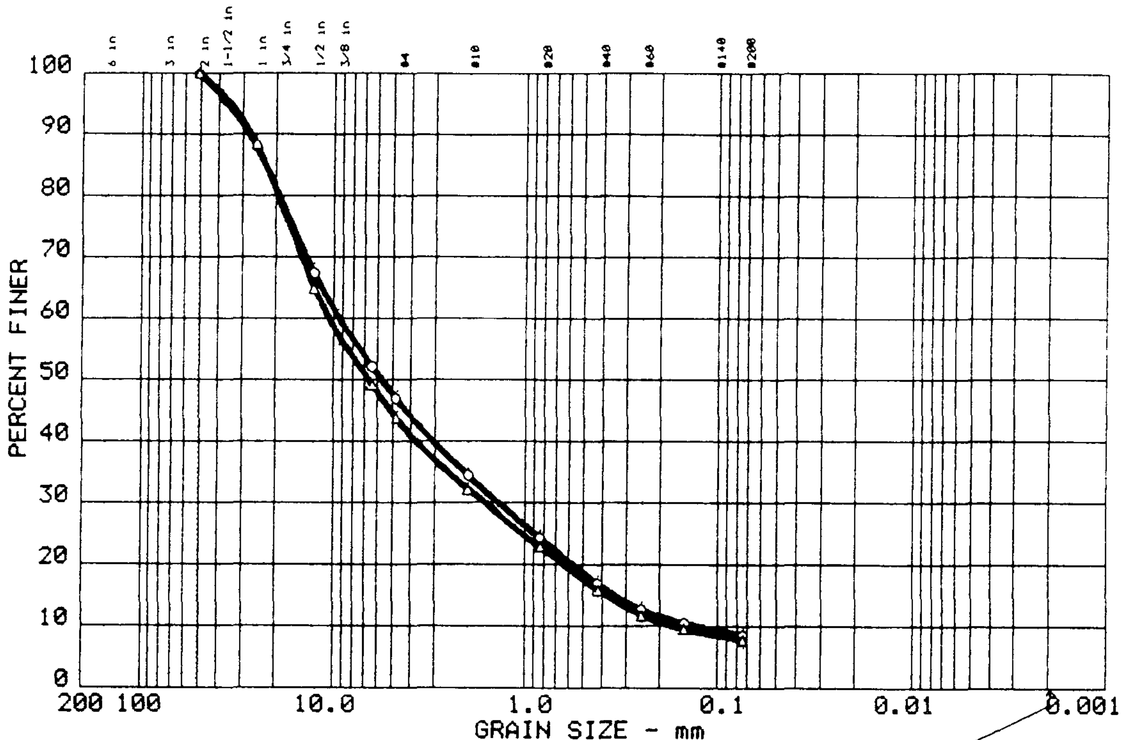
Date: 9-15-90

GRAIN SIZE DISTRIBUTION TEST REPORT

Remarks:

Figure No. 6

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	53.1	38.3	8.6	
△	0.0	56.2	36.1	7.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			22.65	9.33	5.62	1.380	0.3311	0.1259	1.62	74.1
△			22.65	10.59	6.61	1.622	0.3715	0.1679	1.48	63.1

MATERIAL DESCRIPTION	USCS	AASHTO
○ CACL2 - Soil Mix After Compaction	GW-GM	
△ CACL2 - Soil Mix After Compaction	GW-GM	

Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: CACL2 Test Section, STA 1035+50L
△ Location: CACL2 Test Section, STA 1035+75R

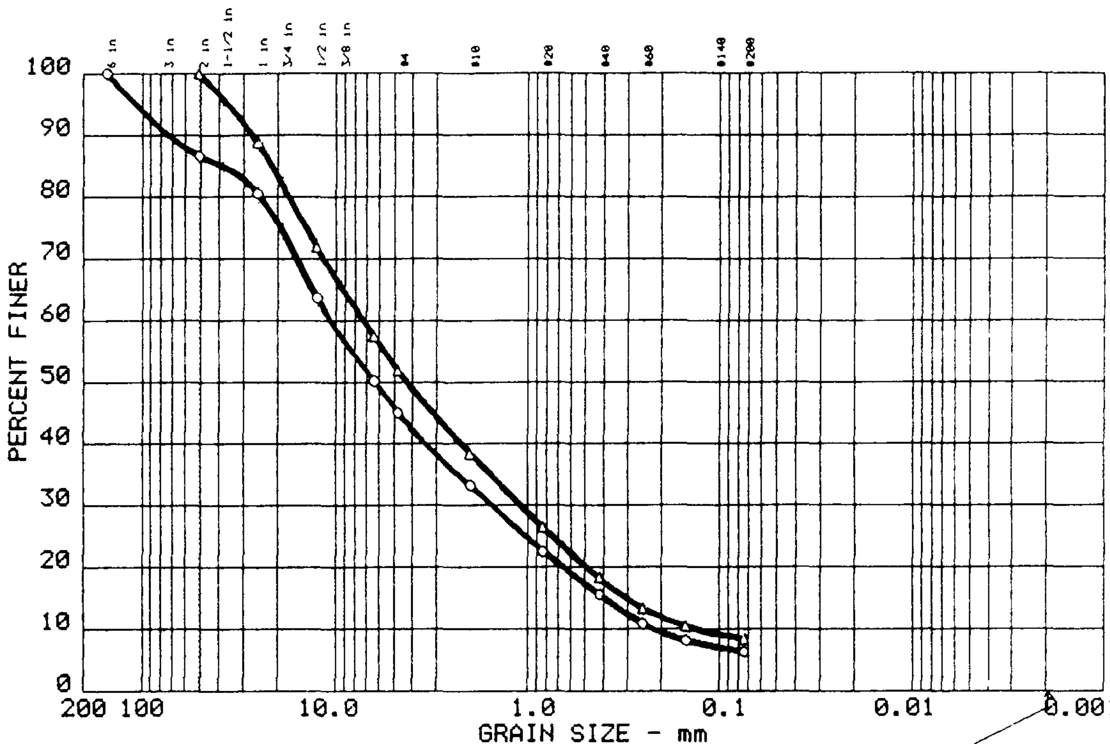
Date: 9-15-90

GRAIN SIZE DISTRIBUTION TEST REPORT

Remarks:

Figure No. 7

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	9.5	45.5	38.6	6.4	
△	0.0	48.1	43.6	8.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			38.90	10.72	6.24	1.531	0.3936	0.2188	1.00	49.0
△			21.55	7.22	4.25	1.093	0.3010	0.1314	1.26	55.0

MATERIAL DESCRIPTION	USCS	AASHTO
○ Standard Subbase Material Below Soil Cement	GP-GM	
△ Standard Subbase Material Below Soil Cement	GW-GM	

Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: Soil Cement Test Section, STA 1028+25
△ Location: Soil Cement Test Section, STA 1029+25

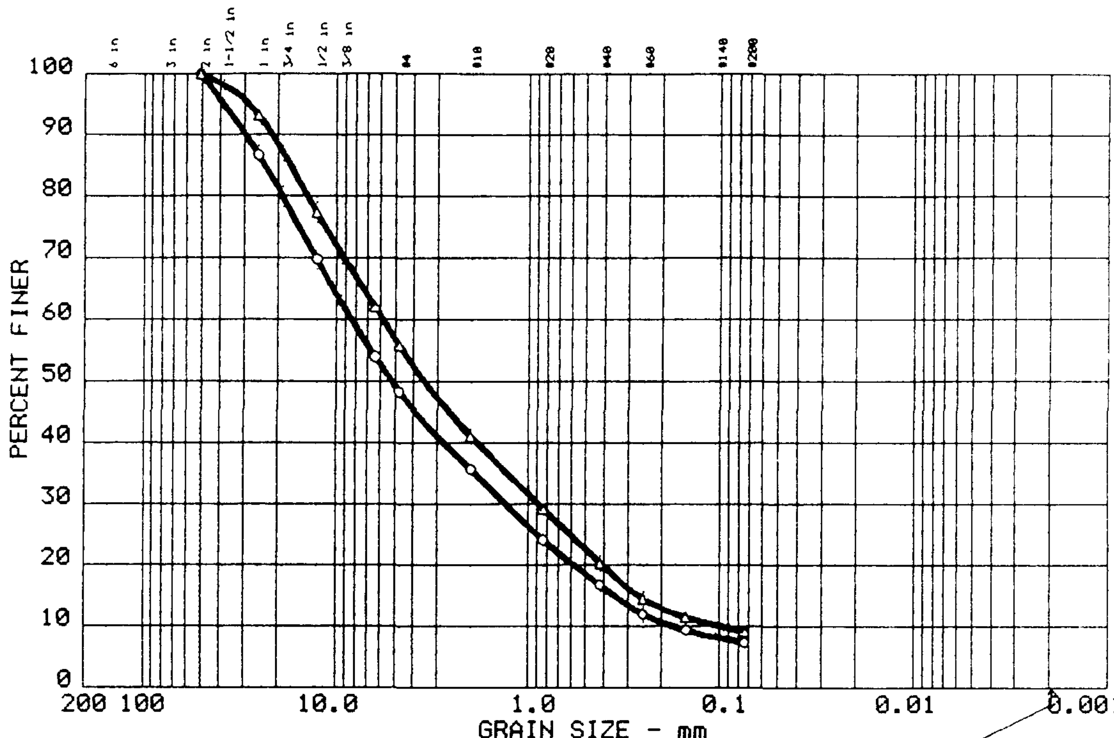
Date: 9-15-90

GRAIN SIZE DISTRIBUTION TEST REPORT

Remarks:

Figure No. 8

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	51.6	41.0	7.4	
△	0.0	44.2	46.6	9.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			23.44	8.39	5.18	1.315	0.3499	0.1754	1.17	47.9
△			17.34	5.74	3.50	0.879	0.2624	0.0931	1.45	61.7

MATERIAL DESCRIPTION	USCS	AASHTO
○ Standard Subbase Material Below Mod. Subbase	GW-GM	
△ Standard Subbase Material Below Mod. Subbase	SW-SM	

Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: Mod. Subbase Section, STA 1030+75
△ Location: Mod. Subbase Section, STA 1031+25

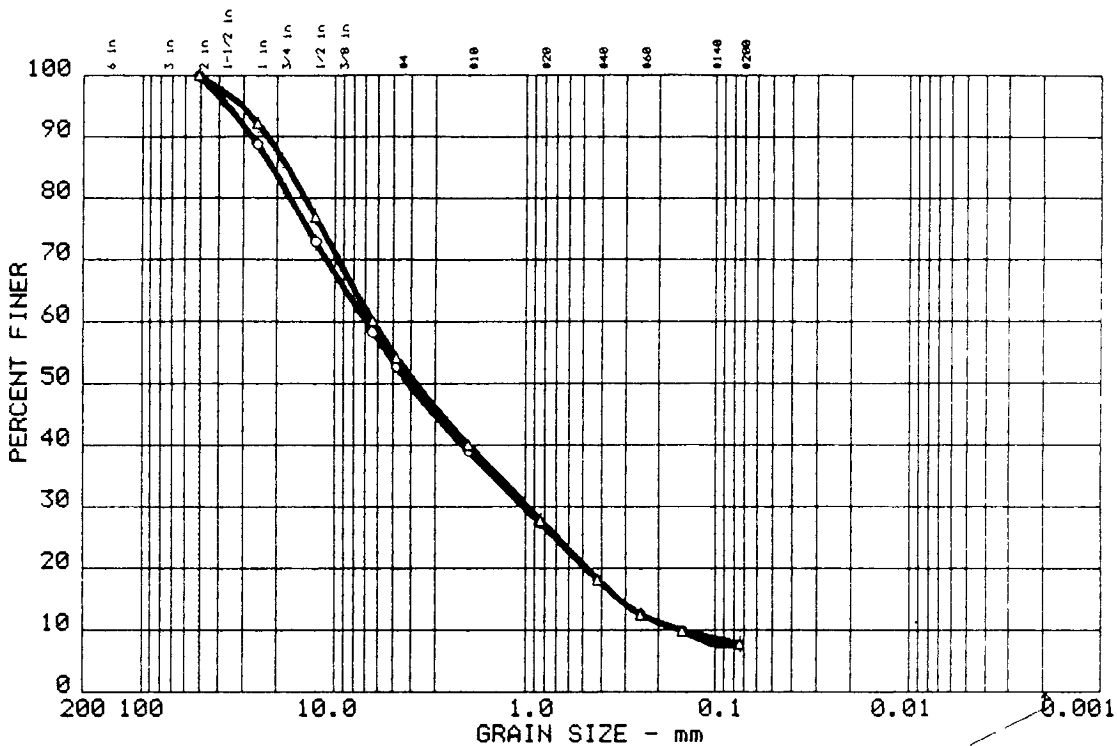
Date: 9-15-90

GRAIN SIZE DISTRIBUTION TEST REPORT

Remarks:

Figure No. 9

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	47.3	45.1	7.6	
△	0.0	45.8	46.4	7.8	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			21.36	6.83	4.07	1.022	0.3195	0.1529	1.00	44.7
△			17.78	6.30	3.75	0.976	0.3195	0.1512	1.00	41.7

MATERIAL DESCRIPTION	USCS	AASHTO
○ Standard Subbase Mat'l. Below Asphalt-Soil Mix	GW-GM	
△ Standard Subbase Mat'l. Below Asphalt-Soil Mix	SP-SM	

Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: Asphalt/Soil Test Section STA 1032+75
△ Location: Asphalt-Soil Test Section STA 1033+25

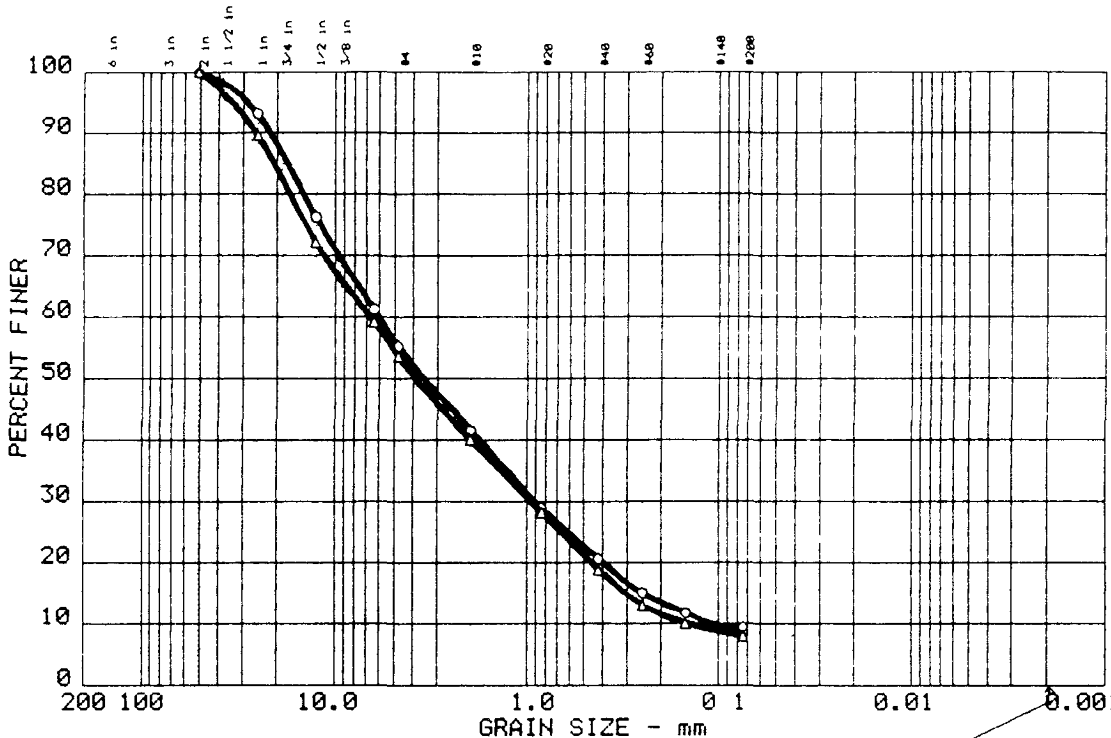
Date: 9-15-90

GRAIN SIZE DISTRIBUTION TEST REPORT

Remarks:

Figure No. 10

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	44.7	45.8	9.5	
△	0.0	46.5	45.6	7.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			17.76	5.95	3.50	0.901	0.2480	0.1095	1.24	54.3
△			20.87	6.60	3.89	0.954	0.3051	0.1444	0.95	45.7

MATERIAL DESCRIPTION	USCS	AASHTO
○ Standard Subbase Material Below CACL2-Soil Mix	SW-SM	
△ Standard Subbase Material Below CACL2-Soil Mix	GP-GM	

Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: CACL2 Test Section, STA 1035+25
△ Location: CACL2 Test Section, STA 1035+75

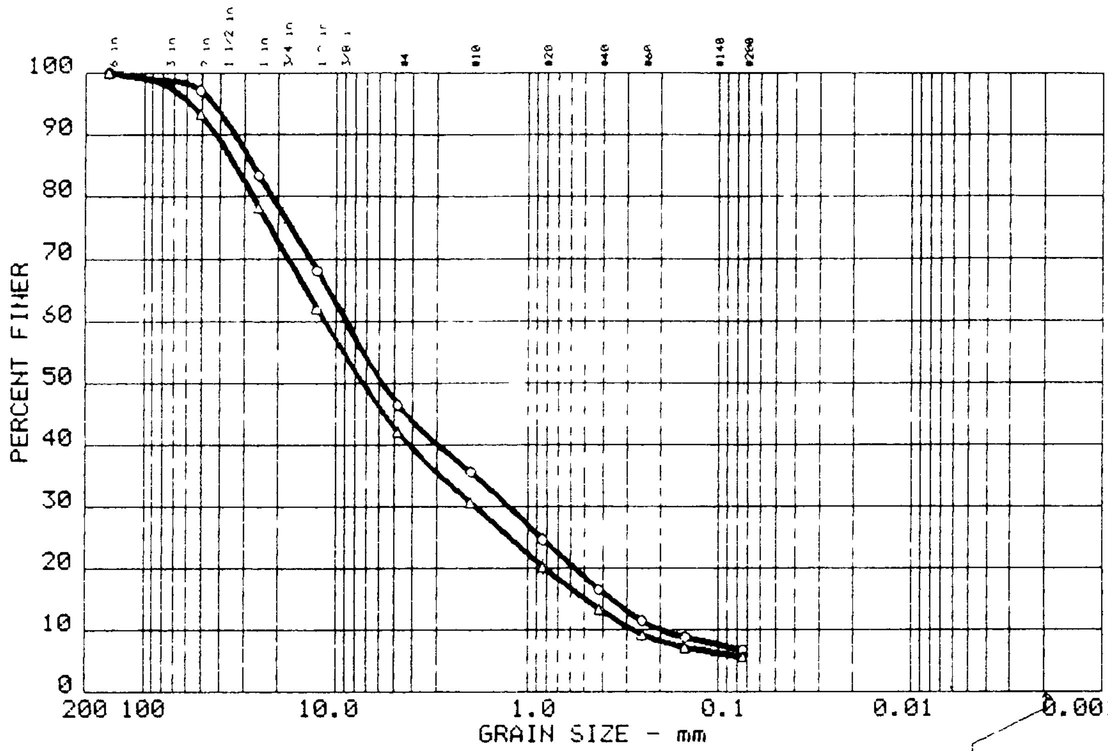
Date: 9-15-90

GRAIN SIZE DISTRIBUTION TEST REPORT

Remarks:

Figure No. 11

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
○	1.3	52.1	39.8	6.8	
△	2.1	55.7	36.6	5.6	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			27.23	8.91	5.69	1.259	0.3589	0.1950	0.91	45.7
△			33.81	11.46	7.23	1.901	0.4943	0.2780	1.14	41.2

MATERIAL DESCRIPTION	USCS	AASHTO
○ Standard Subbase Material	GP-GM	
△ Standard Subbase Material	GW-GM	

Project No.: PA 90-2
Project: MDOT - VAN BUREN, FIELD TRIAL
○ Location: Random Sample, Std. Subbase Cont. Sec
△ Location: Random Sample, Std. Subbase Cont Sec

Date: 9-15-90

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT

Figure No. 12

APPENDIX C

SURVEY CROSS SECTIONS

Legend

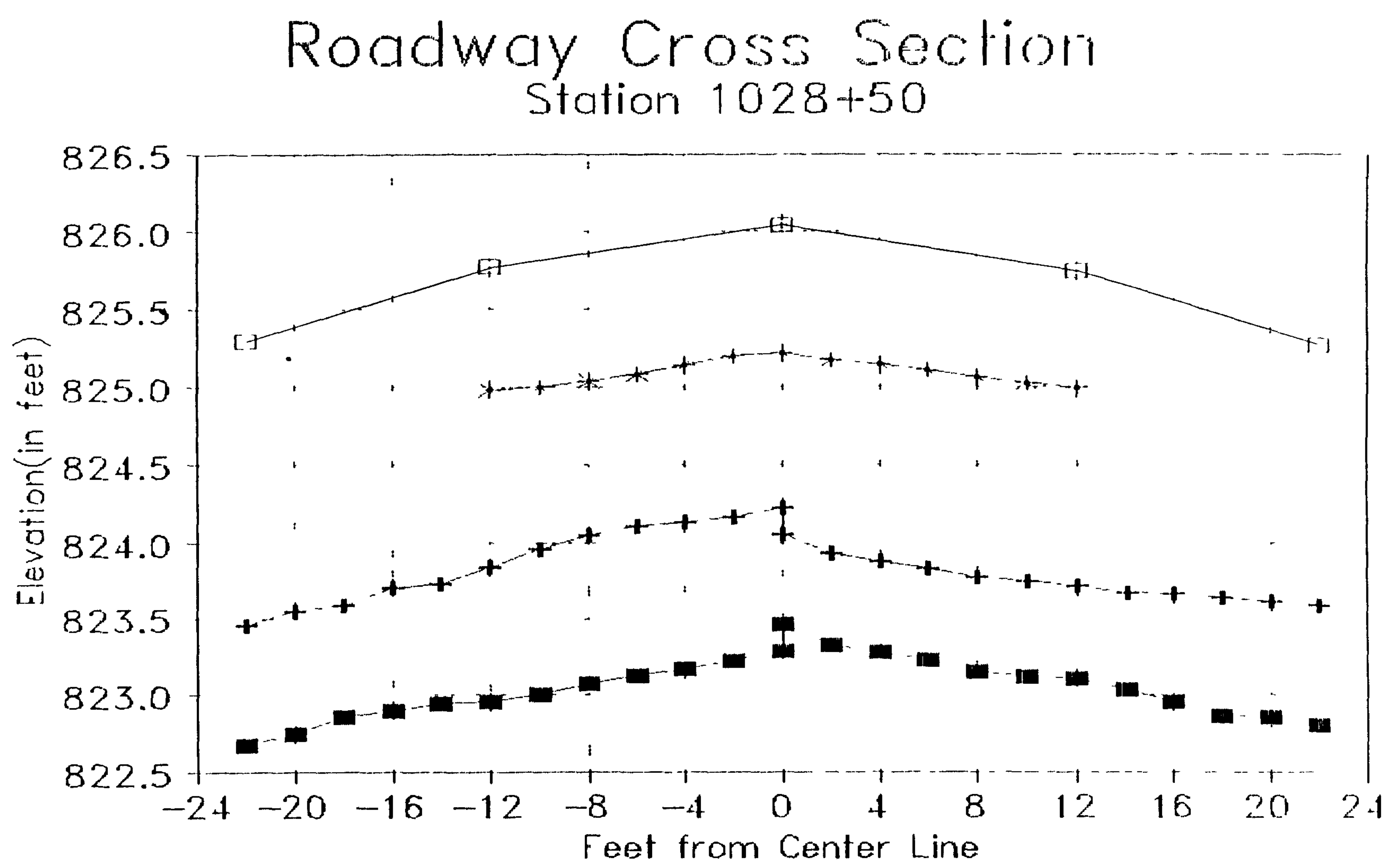
Top of Subgrade

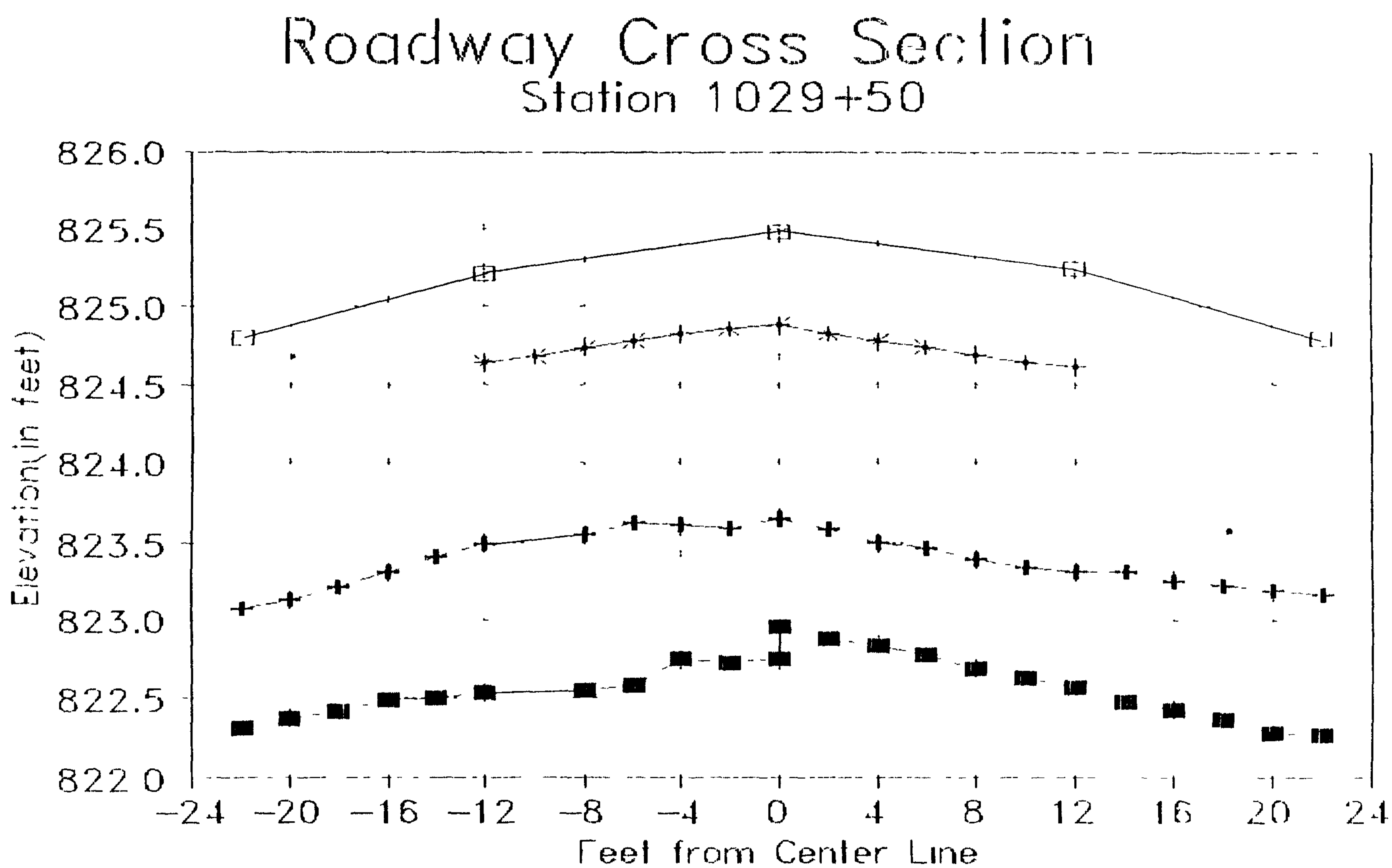
Middle of Subbase

Top of Stabilized or Control Course

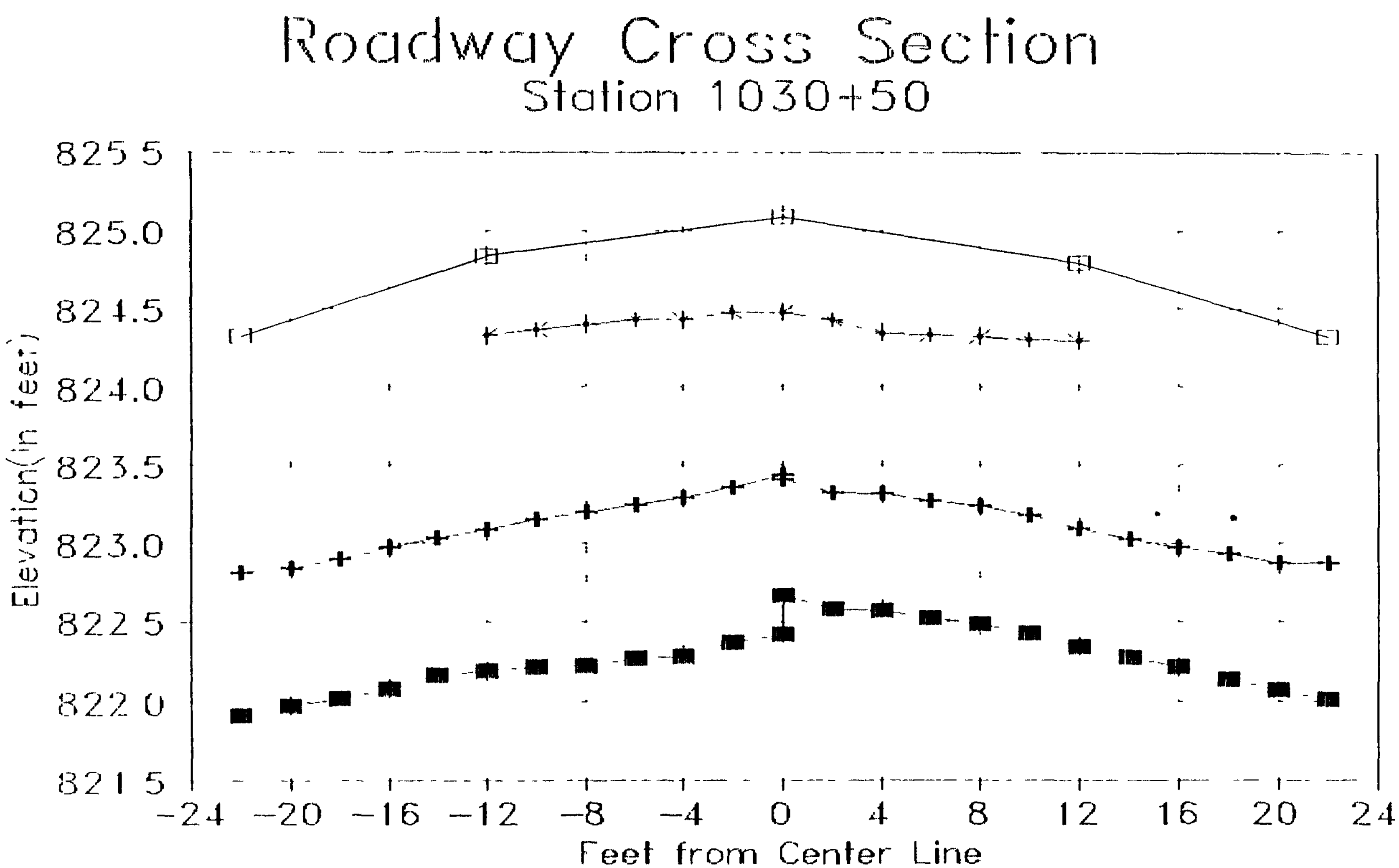
Top of Binder Layer

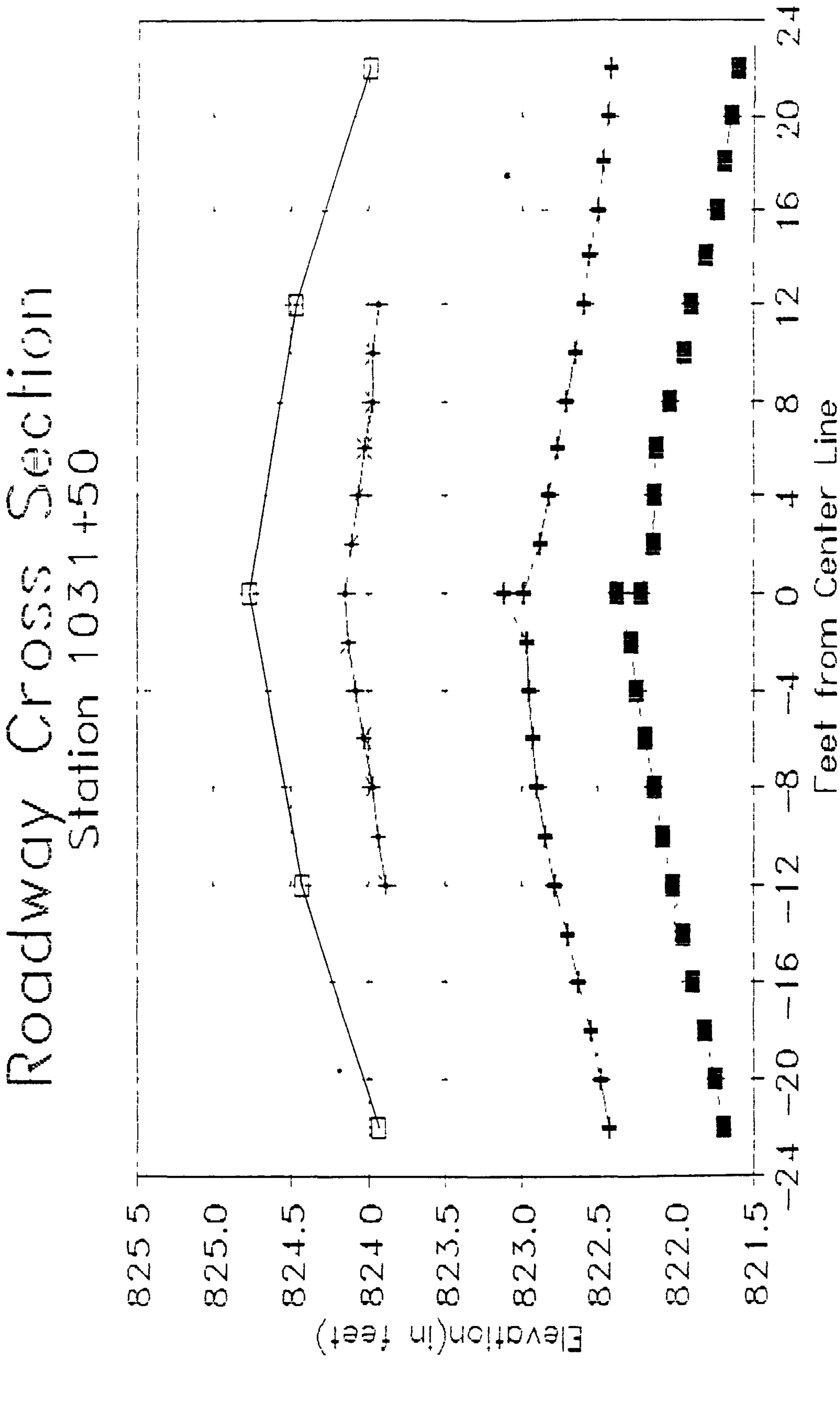
Note: The difference in elevation at the centerline for the "Top of Subgrade" and "Middle of Subbase" measurements is due to construction of the north bound and south bound lanes on different dates.


Roadway Cross Section
Station 1028+50
Elevation(in feet)
826.5
826.0
825.5
825.0
824.5
824.0
823.5
823.0
822.5
−24
−20
−16
−12
−8
−4
0
4
8
12
16
20
24
Feet from Center Line

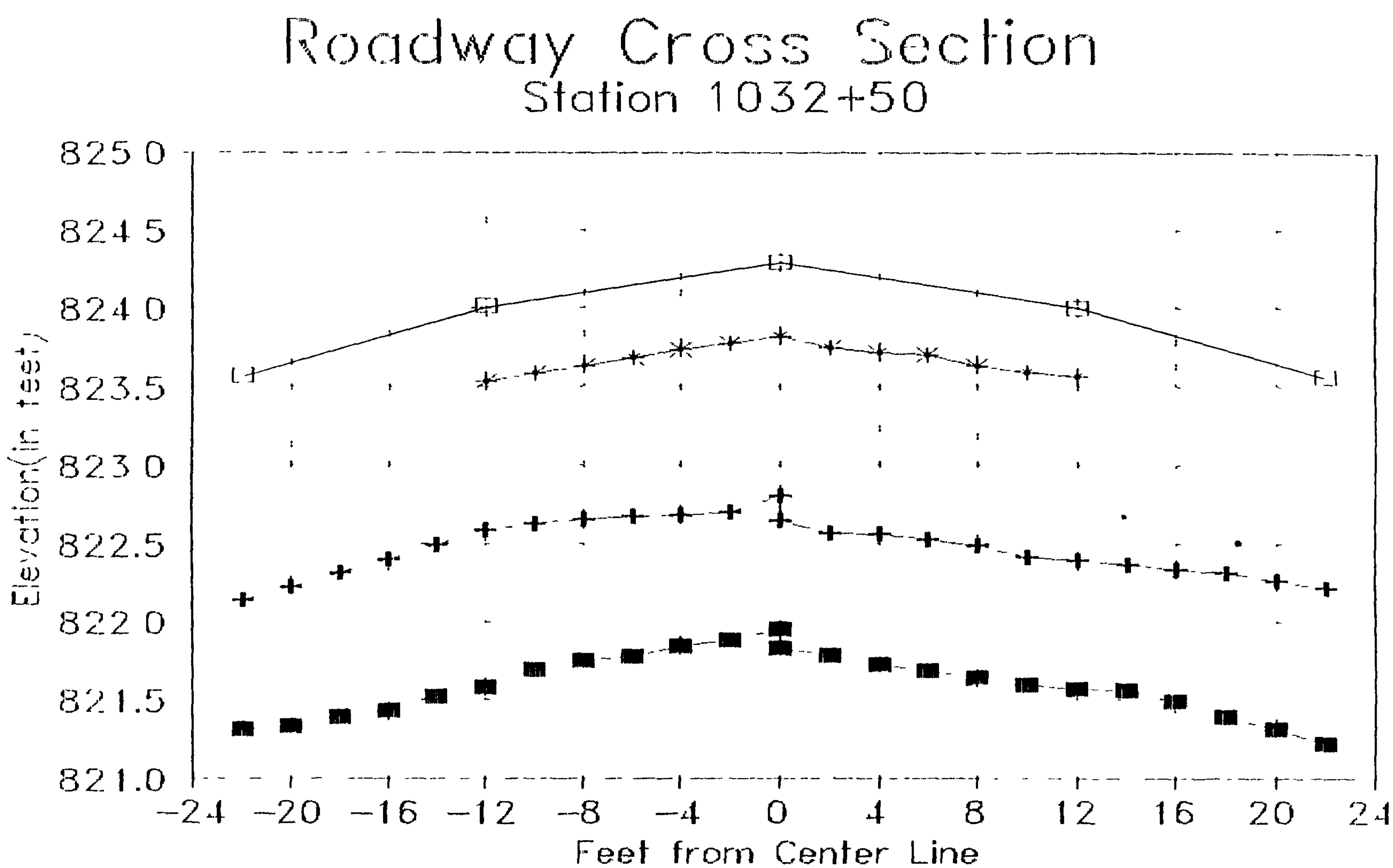


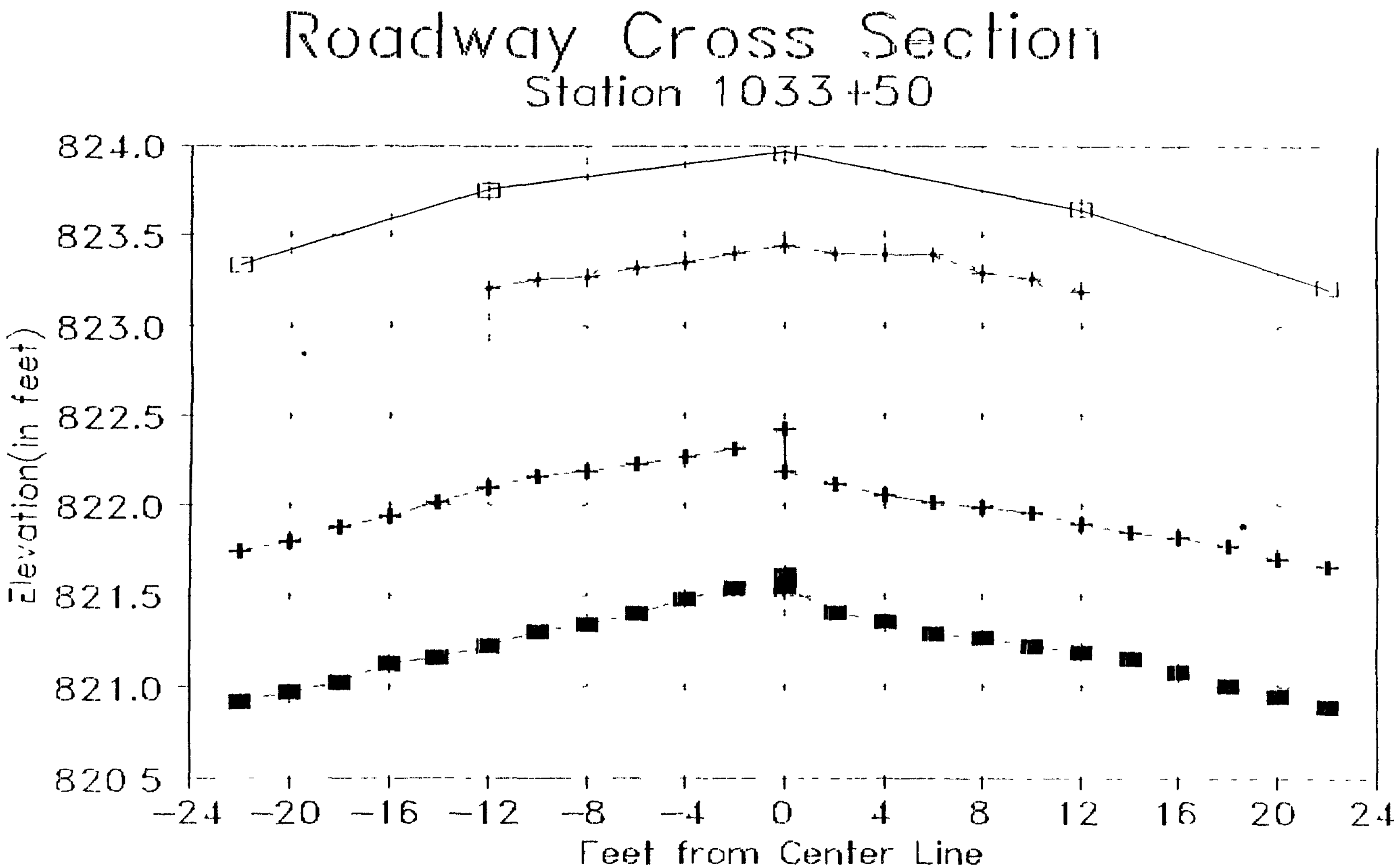
Roadway Cross Section
Station 1029+50
826.0
825.5
825.0
824.5
824.0
823.5
823.0
822.5
822.0
Elevation(in feet)
-24
-20
-16
-12
-8
-4
0
4
8
12
16
20
24
Feet from Center Line

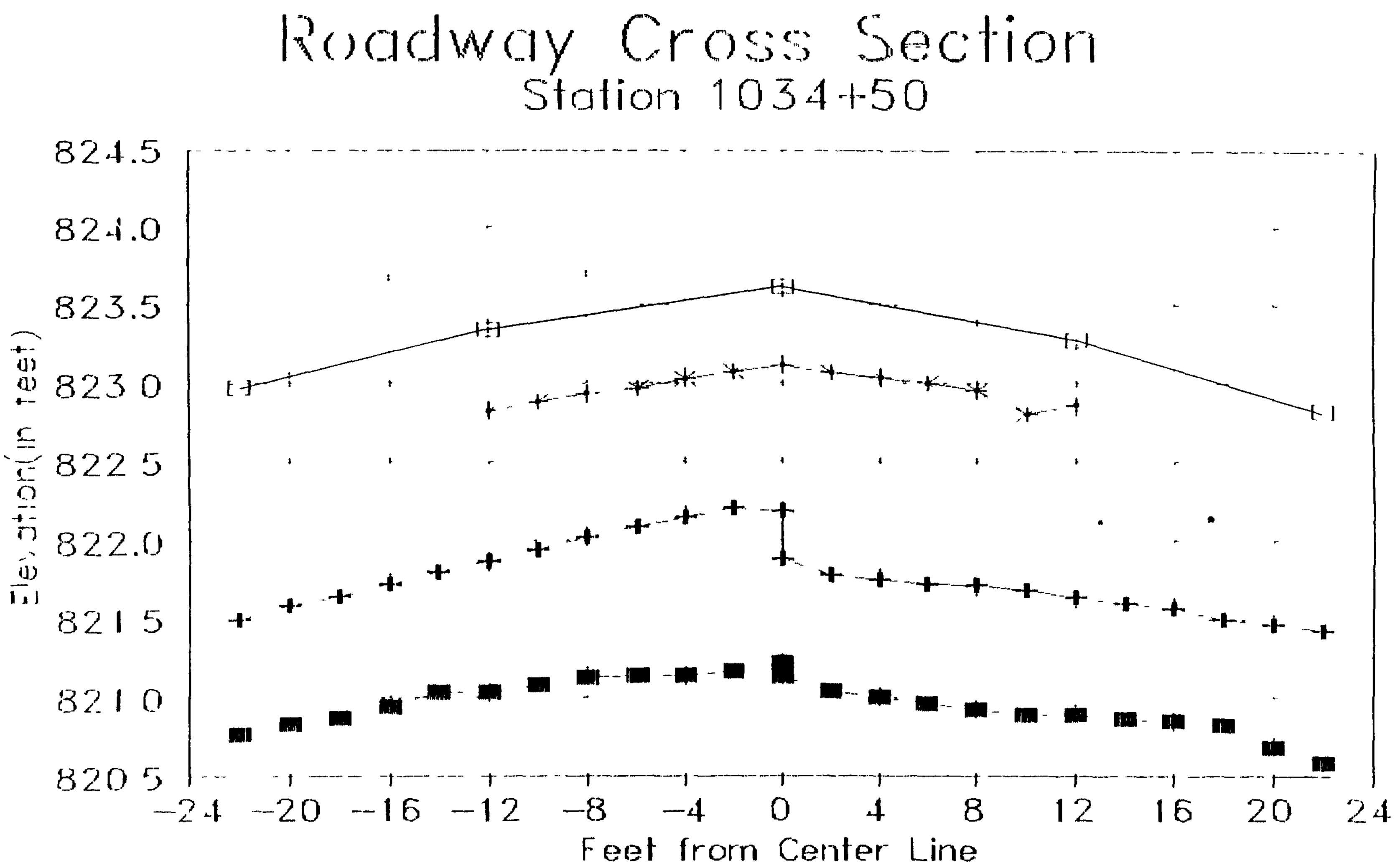

Roadway Cross Section
Station 1030+50
825 5
825.0
824.5
824.0
823.5
823.0
822 5
822 0
821 5
Elevation(in feet)
-24
-20
-16
-12
-8
-4
0
4
8
12
16
20
24
Feet from Center Line



Roadway Cross Section
Station 1031+50
Elevation(in feet)
825.5
825.0
824.5
824.0
823.5
823.0
822.5
822.0
821.5
Feet from Center Line
-24
-20
-16
-12
-8
-4
0
4
8
12
16
20
24


Roadway Cross Section
Station 1032+50
Elevation(in feet)
825.0
824.5
824.0
823.5
823.0
822.5
822.0
821.5
821.0
-24
-20
-16
-12
-8
-4
0
4
8
12
16
20
24
Feet from Center Line

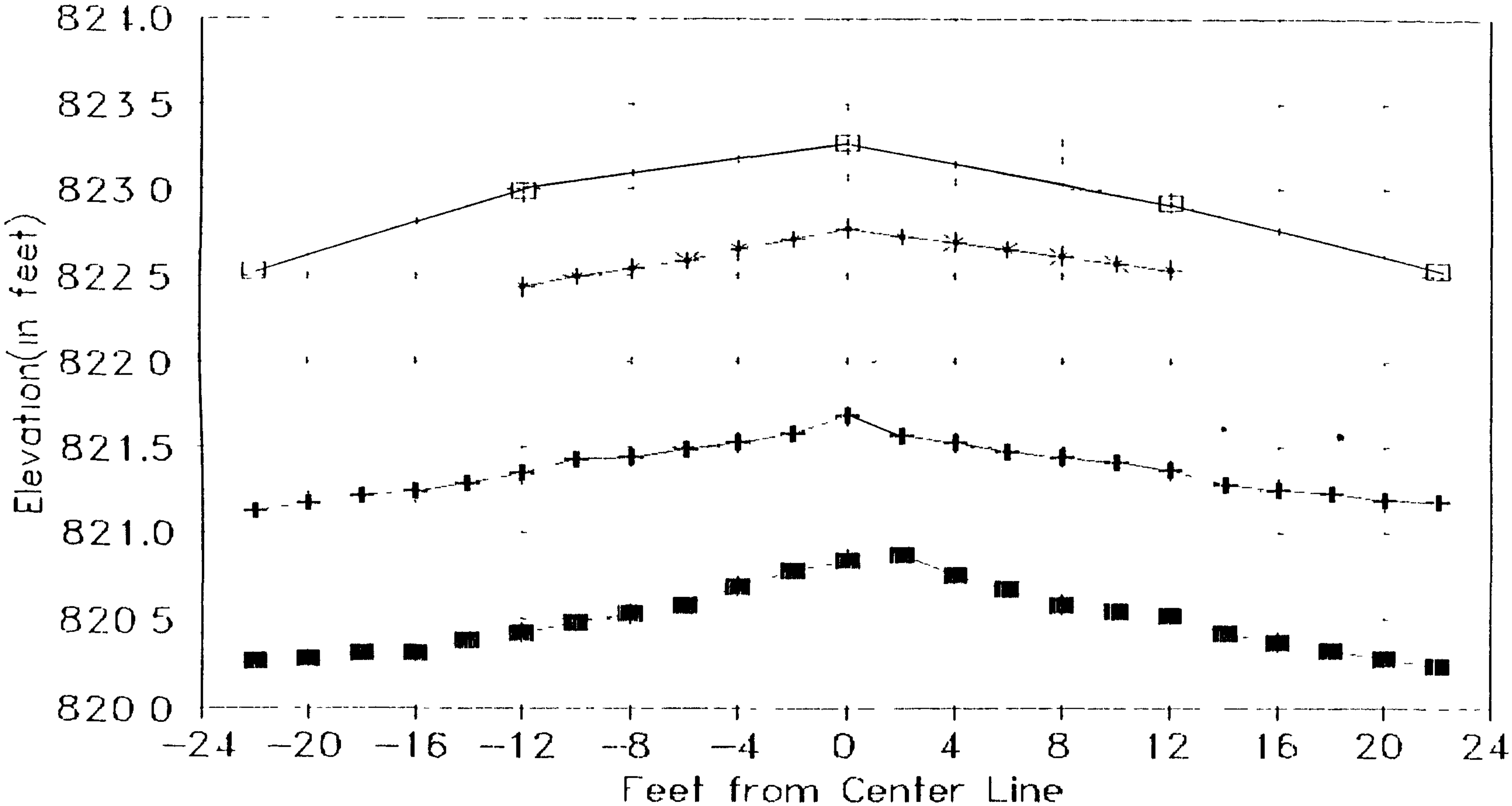

Roadway Cross Section
Station 1033+50
Elevation(in feet)
824.0
823.5
823.0
822.5
822.0
821.5
821.0
820.5
−24
−20
−16
−12
−8
−4
0
4
8
12
16
20
24
Feet from Center Line

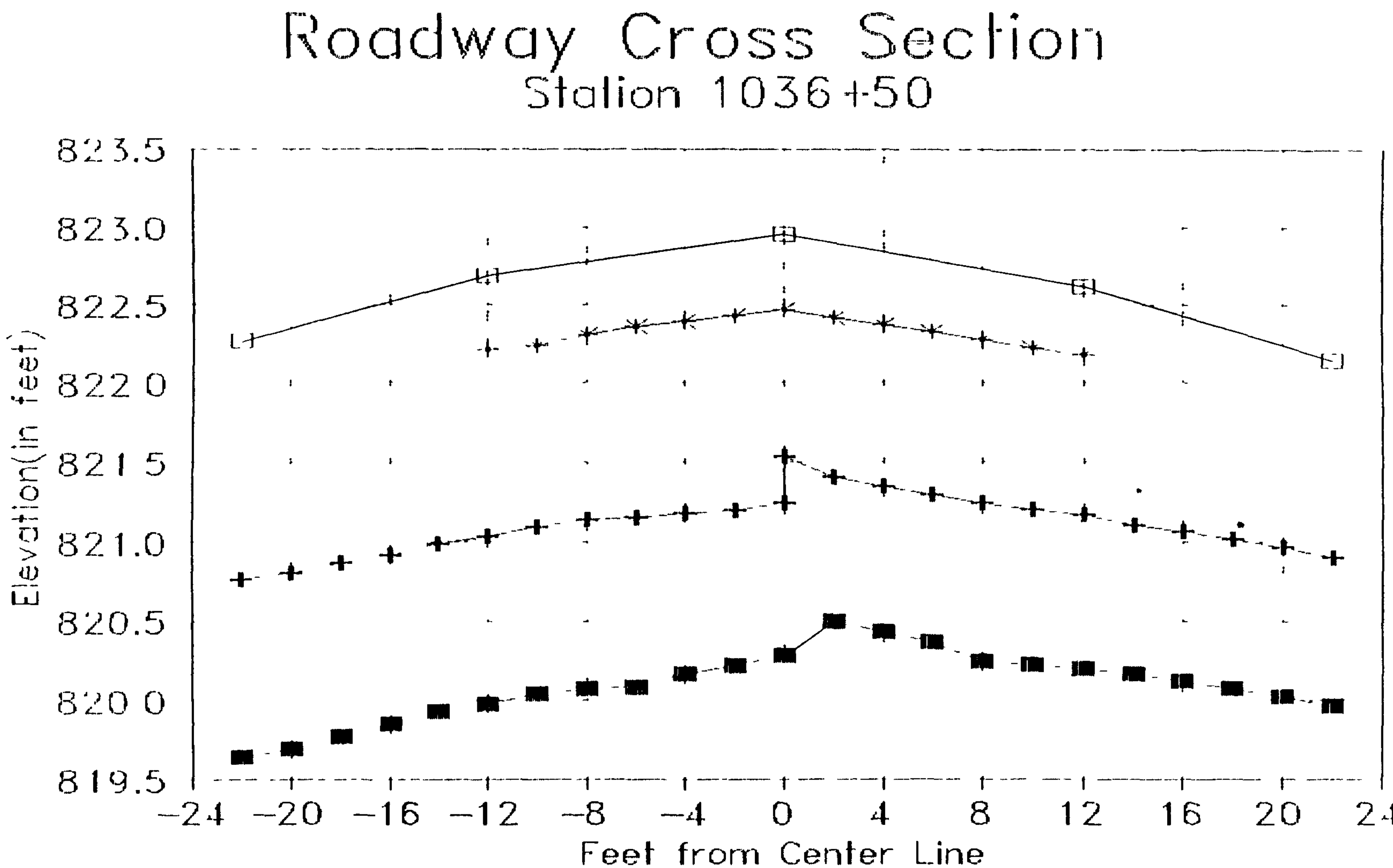


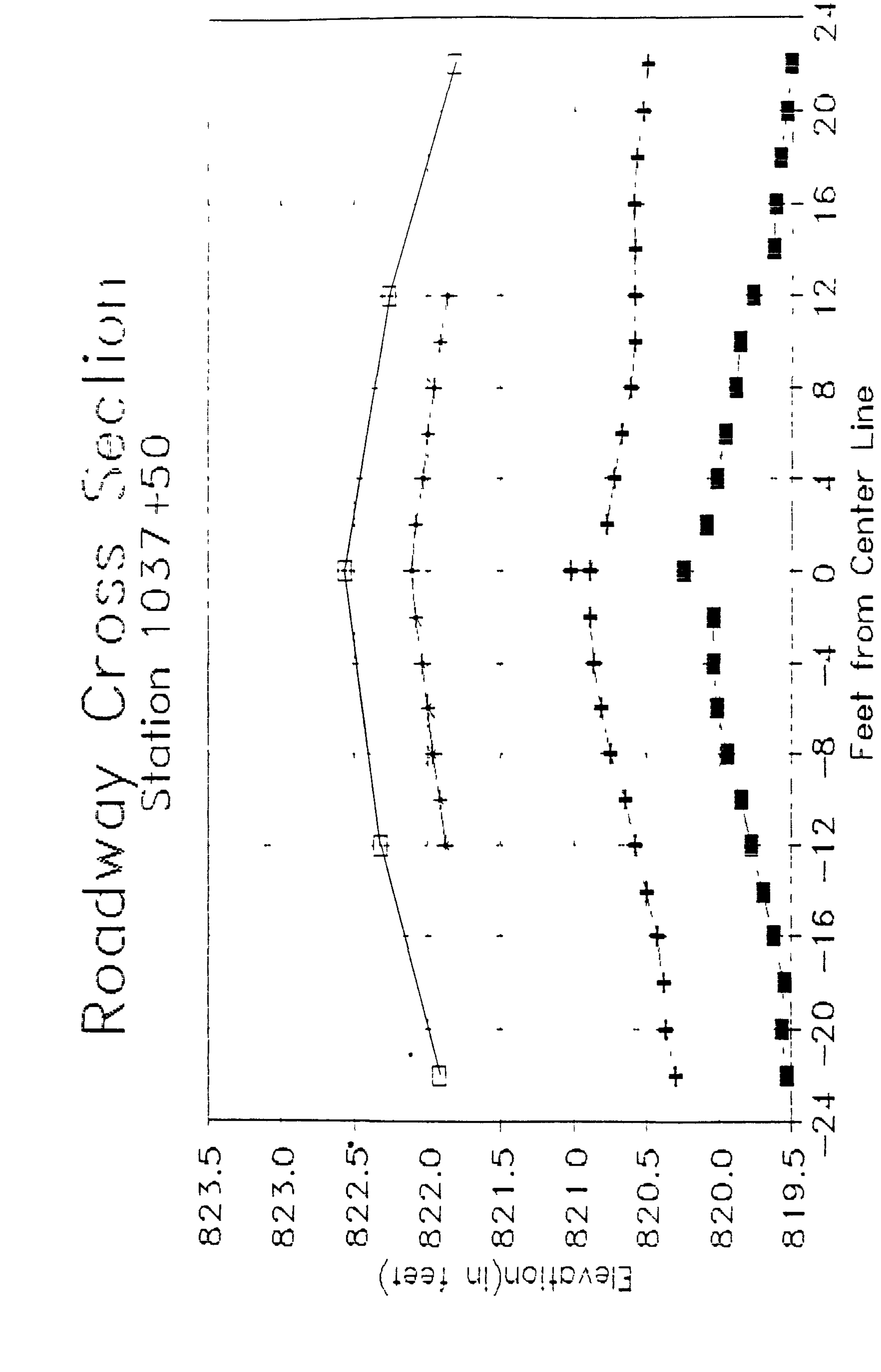
Roadway Cross Section
Station 1034+50
824.5
824.0
823.5
823.0
822.5
822.0
821.5
821.0
820.5
Elevation(in feet)
-24
-20
-16
-12
-8
-4
0
4
8
12
16
20
24
Feet from Center Line

Roadway Cross Section

Station 1035+50




Roadway Cross Section
Station 1036+50
823.5
823.0
822.5
822.0
821.5
821.0
820.5
820.0
819.5
Elevation(in feet)
−24
−20
−16
−12
−8
−4
0
4
8
12
16
20
24
Feet from Center Line



Roadway Cross Section
Station 1037+50
Feet from Center Line
Elevation(in feet)
823.5
823.0
822.5
822.0
821.5
821.0
820.5
820.0
819.5
−24
−20
−16
−12
−8
−4
0
4
8
12
16
20
24

APPENDIX D

ASPHALT STABILIZED BASE TEST RESULTS

Table 1 Preconstruction Marshall stability test results

Asphalt Content (%)	Water Content At Mixing (%)	Unit Weight (pcf)	Marshall Stability (lbs)	Flow Index (0 01")
4 5	3 0	138 4	1146	12 9
4 5	3.0	137 7	1232	9 5
4 5	3 0	138 5	1392	7 7
4 5	3 0	136 5	884	9 5
5 5	3 0	139 7	884	5 7
5 5	3.0	137 3	957	4 5
5 5	3 0	138 9	1117	8 1
5 5	3 0	139 9	754	4 7
5.5	6 0	134 2	378	13 8
5 5	6 0	132 9	247	13 1
5.5	6.0	133 3	319	14.2
5 5	6.0	133 8	261	13 2
5 5	9 0	130.2	116	20 6
5.5	9 0	130 9	101	23 1
5 5	9.0	131 3	160	18.7
5 5	9.0	132 1	174	24 0
6.5	3.0	137 8	667	8.9
6 5	3 0	136 2	725	7 5
6 5	3 0	137 8	566	6 1
6 5	3 0	136 5	508	9 2

Table 2 Marshall stability[a], flow index[b], and unit weight[c] for field compacted samples

Station	Moisture At Mixing (%)[d]	Stability	Flow	Unit Weight
1032+25L	6 6	652	12 9	133 6
1032+25L	6 6	667	9 7	135 5
1032+25L	6 6	479	11 7	132 6
1032+75R	6 6	841	11 8	134 8
1032+75R	6 6	769	8 9	134 0
1032+75R	6 6	812	12 0	135 1
1033+25L	6.6	725	16.2	133 6
1033+25L	6.6	508	19 3	133 3
1033+25L	6 6	537	17 5	131 6
1033+75R	6.6	725	16.0	134 8
1033+75R	6.6	652	10.1	133.6
1033+75R	6.6	827	14.5	135 3

[a] (LBS)
[b] (0.01's inch)
[c] (PCF)
[d] Average Soil Moisture in Field

NOTE The field-generated samples contain an average of 4 5% asphalt by weight of aggregate as determined by MDOT asphalt extraction tests.

Table 3 Marshall stability[a], flow index[b], and unit weight[c] for post-construction lab samples

	Re-Aerated Field Samples			Lab Mixed Samples		
Moisture At Mixing (%)[d]	Stability	Flow	Unit Weight	Stability	Flow	Unit Weight
4 0	1189	4 4	141 5			
4 0	1261	4 4	139 0			
4 0	1450	4.4	140 3			
4 0	1551	3 1	141 6			
4 0	1421	5 7	140.3			
4 0	1769	3.5	141 2			
4 0				1363	4 5	141 9
4 0				1392	4 0	141 6
4 0				1450	5 7	141 5
4 0				1624	6 7	141 7
4 0				1334	6 2	142 4
6.6				884	10.2	137 3
6.6				710	14.5	136 1
6.6				667	13 3	137 2
6 6				798	14 0	136 6
6 6				696	13 8	137 6

[a] (LBS)
[b] (0.01's inch)
[c] (PCF)
[d] Soil Moisture Only

NOTES

1) All of the laboratory generated samples contain 4 5% asphalt by weight of aggregate
2) The re-aerated samples contain an average of 4 5% asphalt by weight of aggregate as determined by MDOT asphalt extraction tests

APPENDIX E

ROAD RATER DEFLECTION MEASUREMENTS

Route #1 Van Buren (Experimental)

```
TEST DOCUMENTATION   FILE UMO3
OPERATOR         [illegible]
DATE             3 2[illegible] 90
TIME             12 15
TEMPERATURE   AT[illegible]
TEST LOCATION
UMO TEST SECTION ASPHALT  TABILI
ZED     BASE

 TEST#    FREQ    FORCE    STIFF
     1    25 0     1 [illegible]     35
 DEFL1   DEFL2    DEFL3    DEFL4
 4 [illegible]    2 5[illegible]       [illegible]4       35
COMMENT STA 1022+25 SBL

 TEST#    FREQ    FORCE    STIFF
     2    25 0     1 50       41
 DEFL1   DEFL2    DEFL3    DEFL4
 3 53    2 2[illegible]       71       24
COMMENT STA 1022+25 SBL

 TEST#    FREQ    FORCE    STIFF
     3    25 0     1 50       30
 DEFL1   DEFL2    DEFL3    DEFL4
 4 30    3 15     1 35       68
COMMENT STA 1022+75 NBL

 TEST#    FREQ    FORCE    STIFF
     4    25 0     1 51       33
 DEFL1   DEFL2    DEFL3    DEFL4
 3 34    2 71     1 24       55
COMMENT STA 1022+75 NBL

    COMPLIANCE----MILS/KIP
TEST# 0    1    2    3    4    5
      +----+----+----+----+----+
 1                 +
 2               +
 3                     +
 4                +
*****       END OF DATA      *****
```

```
TEST DOCUMENTATION   FILE UMO4
OPERATOR         SUC
DATE             3 26 90
TIME             12 30
TEMPERATURE   AT[illegible]
TEST LOCATION
UMO TEST SECTION MODIFIED SUBBAS
E

 TEST#    FREQ    FORCE    STIFF
     1    25 0     1 50       45
 DEFL1   DEFL2    DEFL3    DEFL4
 [illegible] 23    2 13       24       [illegible]
COMMENT STA 1071+25 SBL

 TEST#    FREQ    FORCE    STIFF
     2    25 0     1 50       32
 DEFL1   DEFL2    DEFL3    DEFL4
 4 64    3 04     1 20       43
COMMENT STA 1070+25 SBL

 TEST#    FREQ    FORCE    STIFF
     3    25 0     1 50       35
 DEFL1   DEFL2    DEFL3    DEFL4
 4 26    2 33     1 22       52
COMMENT STA 1070+75 NBL

 TEST#    FREQ    FORCE    STIFF
     4    25 0     1 51       34
 DEFL1   DEFL2    DEFL3    DEFL4
 4 33    2 9[illegible]     1 16       51
COMMENT STA 1071+75 NBL

    COMPLIANCE----MILS/KIP
TEST# 0    1    2    3    4    5
      +----+----+----+----+----+
 1              +
 2                       +
 3                   +
 4                    +
*****       END OF DATA      *****
```

Route #1 Van Buren (Experimental)

```
TEST DOCUMENTATION  FILE UMG1
OPERATOR          [illegible]
DATE              [illegible] 90
TIME              11 45
TEMPERATURE  [illegible]
TEST LOCATION
UMG TEST SECTION STANDARD SUBBAS
E

 TEST#    FREQ    FORCE    STIFF
    1    25 [illegible]   1 5[illegible]    [illegible]0
 DEFL1   DEFL2   DEFL3   DEFL4
 4 5[illegible]   [illegible] 3[illegible]   1 [illegible]     5[illegible]
COMMENT STA 10[illegible]7+25 SBL

 TEST#    FREQ    FORCE    STIFF
    2    25 0    1 5[illegible]    [illegible]
 DEFL1   DEFL2   DEFL3   DEFL4
 4 5[illegible]   [illegible]   1 16     41
COMMENT STA 1026+25 SBL

 TEST#    FREQ    FORCE    STIFF
    3    25 [illegible]   1 51     [illegible]4
 DEFL1   DEFL2   DEFL3   DEFL4
 4 [illegible]9   3 15    1 31     [illegible]
COMMENT STA [illegible]6+75 NBL

 TEST#    FREQ    FORCE    STIFF
    4    25.0    1 50     36
 DEFL1   DEFL2   DEFL3   DEFL4
 4 16   [illegible] 97    1 24     46
COMMENT STP 1037+75 NBL

    COMPLIANCE----MILS KIF
TEST# 0    1    2    3    4    5
      +----+----+----+----+----+
 1                       +
 2                     +
 3                     +
 4
*****        END OF DATA      *****
```

```
TEST DOCUMENTATION  FILE UMG2
OPERATOR          [illegible]
DATE              [illegible] 90
TIME              1[illegible] [illegible]0
TEMPERATURE  [illegible]
TEST LOCATION
UMG TEST SECTION [illegible] STABILIZE
D    BASE

 TEST#    FREQ    FORCE    STIFF
    1    [illegible]   [illegible] 51     46
 DEFL1   DEFL2   DEFL3   DEFL4
 [illegible] 24   [illegible] 23     33      43
COMMENT STA 1035+40 SBL

 TEST#    FREQ    FORCE    STIFF
    2    25 0    1 50     41
 DEFL1   DEFL2   DEFL3   DEFL4
 [illegible] 60   [illegible] 42     36      41
COMMENT STA 1034+40 SBL

 TEST#    FREQ    FORCE    STIFF
    3    25 0    1 50     30
 DEFL1   DEFL2   DEFL3   DEFL4
 4 33   [illegible] 69   1 33    1 00
COMMENT STA 1035+25 NBL

 TEST#    FREQ    FORCE    STIFF
    4    25 0    1 51     30
 DEFL1   DEFL2   DEFL3   DEFL4
 5 03   [illegible] 62   1 27    1 09
COMMENT STA 1035+75 NBL

    COMPLIANCE----MILS KIF
TEST# 0    1    2    3    4    5
      +----+----+----+----+----+
 1                +
 2                 +
 3                       +
 4                       +
*****        END OF DATA      *****
```

Route #1 Van Buren (Experimental)

```
TEST DOCUMENTATION  FILE UMO5
OPERATOR        [illegible]
DATE            3 22 90
TIME            1[illegible] 45
TEMPERATURE     [illegible]
TEST LOCATION
UMO TEST SECTION SOIL-CEMENT STA
BILIZED     BASE

 TEST#    FREQ    FORCE    STIFF
   [illegible]    [illegible]5 [illegible]   [illegible]  [illegible]      [illegible]
 DEFL1   DEFL[illegible]   DEFL[illegible]   DEFL4
  1 [illegible]    [illegible] [illegible]      [illegible]       50
COMMENT STA [illegible]0[illegible]9+[illegible]5 [illegible]EL

 TEST#    FREQ    FORCE    STIFF
    [illegible]    [illegible]5 [illegible]    1 [illegible]       [illegible]1
 DEFL1   DEFL[illegible]   DEFL[illegible]   DEFL4
  [illegible] 10    1 63    1 1[illegible]      56
COMMENT STA 1028+[illegible]5 [illegible]EL

 TEST#    FREQ    FORCE    STIFF
    [illegible]    [illegible] 1    1 [illegible]       57
 DEFL1   DEFL2   DEFL[illegible]   DEFL4
  2 6[illegible]    [illegible] 13    1 40       74
COMMENT STA 1028+75 NBL

 TEST#    FREQ    FORCE    STIFF
     4    [illegible]5 [illegible]    1 50       57
 DEFL1   DEFL2   DEFL[illegible]   DEFL4
  2 6[illegible]    [illegible] 13    1 4[illegible]      7[illegible]
COMMENT STA 10[illegible]9+75 NEL

    COMPLIANCE----MILS/KIP
TEST# 0     1     2     3     4     5
      +-----+-----+-----+-----+-----+
 1           +
 2            +
 3               +
 4               +
*****        END OF DATA        *****
```

```
TEST DOCUMENTATION  FILE UMO7
OPERATOR        SUC
DATE            5 21 91
TIME            11 45
TEMPERATURE   A77F35
TEST LOCATION
UMO TEST SECTION ASPHALT STABILI
ZED     BASE

 TEST#    FEED    FORCE    STIFF
     1    25 0     1 43       64
 DEFL1   DEFL2   DEFL3    DEFL4
  2 31    1 66      35       4
COMMENT STA 1022+25 SEL

 TEST#    FEED    FORCE    STIFF
     2    25 0     1 43       60
 DEFL1   DEFL2   DEFL3    DEFL4
  2 47    1 75      33       27
COMMENT STA 1022+25 SEL

 TEST#    FEED    FORCE    STIFF
     3    25 0     1 50       46
 DEFL1   DEFL2   DEFL3    DEFL4
  3 24    2 71      37       50
COMMENT STA 1022+75 HEL

 TEST#    FEED    FORCE    STIFF
     4    25 0     1 50       43
 DEFL1   DEFL2   DEFL3    DEFL4
  3 12    2 27     1 33      67
COMMENT STA 1022+75 HEL

   COMPLIANCE----MILS KIP
TEST# 0     1     2     3     4     5
      +-----+-----+-----+-----+-----+
1               +
2               +
3                   +
4                  +
*****      END OF DATA      *****
```

```
TE-T DOCUMENTATION  FILE UMO4
OPERATOR        U
DATE            5 21 91
TIME            12 00
TEMPERATURE   A77F35
TEST LOCATION
UMO TEST SECTION MODIFIED SUBBAS
E

 TEST#    FEED    FORCE    STIFF
     1    25 0     1 50       40
 DEFL1   DEFL2   DEFL3    DEFL4
  3 75    2 67     1 24      43
COMMENT STA 1071+25 SEL

 TEST#    FEED    FORCE    STIFF
     2    25 0     1 50       41
 DEFL1   DEFL2   DEFL3    DEFL4
  3 65    2 62      62       53
COMMENT STA 1070+25 SEL

 TEST#    FEED    FORCE    STIFF
     3    25 0     1 51       33
 DEFL1   DEFL2   DEFL3    DEFL4
  3 36    2 33     1 57      75
COMMENT STA 1070+75 HEL

 TEST#    FEED    FORCE    STIFF
     4    25 0     1 50       37
 DEFL1   DEFL2   DEFL3    DEFL4
  3 37    2 64     1 30      57
COMMENT STA 1071+75 HEL

   COMPLIANCE----MILS KIP
TEST# 0     1     2     3     4     5
      +-----+-----+-----+-----+-----+
1                      +
2                     +
3                      +
4                       +
*****      END OF DATA      *****
```

```
TEST DOCUMENTATION   FILE 1[illegible]1
OPERATOR          U[illegible]
DATE              5 21 81
TIME              11 15
TEMPERATURE    A77F35
TEST LOCATION
UMO TEST SECTION STANDARD SUBBAS
E

  TEST#     FREQ     FORCE     STIFF
     1      25 0     1 51       73
 DEFL1    DEFL2    DEFL3     DEFL4
  3 37     2 45     1 35       51
COMMENT  STA 1077+25 SEL

  TEST#     FREQ     FORCE     STIFF
     2      25 0     1 53       52
 DEFL1    DEFL2    DEFL3     DEFL4
  3 52     2 37      93        44
COMMENT  STA 1076+25 SEL

  TEST#     FREQ     FORCE     STIFF
     3      25 0     1 50       75
 DEFL1    DEFL2    DEFL3     DEFL4
  4 17     2 54     1 37       64
COMMENT  STA 1076+75 NEL

  TEST#     FREQ     FORCE     STIFF
     4      25 0     1 51       41
 DEFL1    DEFL2    DEFL3     DEFL4
  3 57     2 50     1 02       51
COMMENT  STA 1077+75 NEL

      COMPLIANCE----MILS KIP
TEST# 0     1     2     3     4     5
      +-----+-----+-----+-----+-----+
 1                      +
 2                +
 3                         +
 4                      +
*****        END OF DATA        *****
```

```
TEST DOCUMENTATION   FILE UMO2
OPERATOR          SUC
DATE              5 21 81
TIME              11 30
TEMPERATURE    A77F35
TEST LOCATION
UMO TEST SECTION CALL STABILIZE
D     BASE

  TEST#     FREQ     FORCE     STIFF
     1      25 0     1 52       62
 DEFL1    DEFL2    DEFL3     DEFL4
  2 42     1 73     1 00       45
COMMENT  STA 1075+40 SEL

  TEST#     FREQ     FORCE     STIFF
     2      25 0     1 50       56
 DEFL1    DEFL2    DEFL3     DEFL4
  2 67     1 37      62        34
COMMENT  STA 1074+40 SEL

  TEST#     FREQ     FORCE     STIFF
     3      25 0     1 51       36
 DEFL1    DEFL2    DEFL3     DEFL4
  4 17     3 01     1 25      1 11
COMMENT  STA 1075+25 NEL

  TEST#     FREQ     FORCE     STIFF
     4      25 0     1 51       42
 DEFL1    DEFL2    DEFL3     DEFL4
  3 57     2 63     1 70      1 12
COMMENT  STA 1075+75 NEL

      COMPLIANCE----MILS KIP
TEST# 0     1     2     3     4     5
      +-----+-----+-----+-----+-----+
 1              +
 2               +
 3                      +
 4                   +
*****        END OF DATA        *****
```

```
TEST DOCUMENTATION  FILE 10
OPERATOR          SUC
DATE              5 21 81
TIME              12 15
TEMPERATURE   A77F85
TEST LOCATION
UMO TEST SECTION SOIL-CEMENT STA
STILIZED     BASE

 TEST#    FREQ    FORCE     STIFF
     1    25 3     1 51        72
 DEFL1   DEFL2    DEFL3     DEFL4
  2 06    1 63     1 11        53
COMMENT STA 1023+25 SEL

 TEST#    FREQ    FORCE     STIFF
     2    25 0     1 43        62
 DEFL1   DEFL2    DEFL3     DEFL4
  2 35    1 33     1 24        57
COMMENT STA 1023+25 SEL

 TEST#    FREQ    FORCE     STIFF
     3    25 3     1 51        50
 DEFL1   DEFL2    DEFL3     DEFL4
  3 02    2 46     1 63        32
COMMENT STA 1023+75 MEL

 TEST#    FREQ    FORCE     STIFF
     4    25 0     1 50        56
 DEFL1   DEFL2    DEFL3     DEFL4
  2 64    2 11       90        37
COMMENT STA 1023+75 MEL

    COMPLIANCE----MILS / IP
TEST# 0    1    2    3    4    5
      +----+----+----+----+----+
 1           +
 2             +
 3                 +
 4               +
*****        END OF DATA      *****
```

```
TEST DOCUMENTATION  FILE UM01
OPERATOR        SWC
DATE            9 6 81
TIME            12 15
TEMPERATURE   A75F59
TEST LOCATION
UMO TEST SECTION STANDARD SUBBAS
E

 TEST#    FREQ    FORCE    STIFF
     1    25 9     1 50       55
 DEFL1   DEFL2    DEFL3    DEFL4
  2 72    1 43       30       33
COMMENT STA 1037+25 SBL

 TEST#    FREQ    FORCE    STIFF
     2    25 0     1 50       33
 DEFL1   DEFL2    DEFL3    DEFL4
  1 51    1 05       62       30
COMMENT STA 1036+25 SBL

 TEST#    FREQ    FORCE    STIFF
     3    25 8     1 51       76
 DEFL1   DEFL2    DEFL3    DEFL4
  1 99    1 33       73       42
COMMENT STA 1036+75 NBL

 TEST#    FREQ    FORCE    STIFF
     4    25 9     1 51       77
 DEFL1   DEFL2    DEFL3    DEFL4
  1 94    1 30       73       35
COMMENT STA 1037+75 NBL

    COMPLIANCE----MILS/KIP
TEST# 0    1    2    3    4    5
      +----+----+----+----+----+
 1              +
 2         +
 3           +
 4          +
*****      END OF DATA      *****
```

```
TEST DOCUMENTATION  FILE UM02
OPERATOR        SWC
DATE            9 6/81
TIME            12 30
TEMPERATURE   A75F58
TEST LOCATION
UMO TEST SECTION CACL2 STABILIZE
D SUBBASE

 TEST#    FREQ    FORCE    STIFF
     1    25 0     1 52       31
 DEFL1   DEFL2    DEFL3    DEFL4
  1 67    1 12       66       35
COMMENT STA 1035+40 SBL

 TEST#    FREQ    FORCE    STIFF
     2    25 0     1 50       52
 DEFL1   DEFL2    DEFL3    DEFL4
  1 92    1 11       66       32
COMMENT STA 1034+40 SBL

 TEST#    FREQ    FORCE    STIFF
     3    24 9     1 49       64
 DEFL1   DEFL2    DEFL3    DEFL4
  2 32    1 70     1 12       70
COMMENT STA 1035+25 NBL

 TEST#    FREQ    FORCE    STIFF
     4    25 9     1 51       63
 DEFL1   DEFL2    DEFL3    DEFL4
  2.38    1 83     1 27       62
COMMENT STA 1035+75 NBL

    COMPLIANCE----MILS/KIP
TEST# 0    1    2    3    4    5
      +----+----+----+----+----+
 1         +
 2          +
 3            +
 4            +
*****      END OF DATA      *****
```

```
TEST DOCUMENTATION, FILE UMO3
OPERATOR       BWC
DATE           8/6/91
TIME           12 45
TEMPERATURE    A75F88
TEST LOCATION
UMO TEST SECTION ASPHALT STABILI
ZED SUBBASE

 TEST#    FREQ    FORCE    STIFF
     1    25 0     1 52     1 09
 DEFL1   DEFL2    DEFL3    DEFL4
  1 40      99       60       39
COMMENT STA 1033+25 SBL

 TEST#    FREQ    FORCE    STIFF
     2    25 0     1 50     1 11
 DEFL1   DEFL2    DEFL3    DEFL4
  1 34      92       52       23
COMMENT STA 1032+25 SBL

 TEST#    FREQ    FORCE    STIFF
     3    25 0     1 50       91
 DEFL1   DEFL2    DEFL3    DEFL4
  1 33    1 29       89       51
COMMENT STA 1032+75 NBL

 TEST#    FREQ    FORCE    STIFF
     4    25 0     1 52       83
 DEFL1   DEFL2    DEFL3    DEFL4
  1 81    1 28       77       40
COMMENT STA 1033+75 NBL

      COMPLIANCE----MILS/KIP
TEST# 0     1     2     3     4     5
      +-----+-----+-----+-----+-----+
 1          +
 2          +
 3           +
 4           +
*****       END OF DATA       *****
```

```
TEST DOCUMENTATION  FILE UMO4
OPERATOR       BWC
DATE           8 6/91
TIME           1 00
TEMPERATURE    A75F88
TEST LOCATION
UMO TEST SECTION MODIFIED SUBBAS
E

 TEST#    FREQ    FORCE    STIFF
     1    25 0     1 51       92
 DEFL1   DEFL2    DEFL3    DEFL4
  1 34    1 13       59       27
COMMENT STA 1031+25 SBL

 TEST#    FREQ    FORCE    STIFF
     2    25 0     1 51       72
 DEFL1   DEFL2    DEFL3    DEFL4
  2 09    1 30       63       40
COMMENT 1030+25 SBL

 TEST#    FREQ    FORCE    STIFF
     3    25 0     1 50       72
 DEFL1   DEFL2    DEFL3    DEFL4
  2 09    1 46       85       43
COMMENT STA 1030+75 NBL

 TEST#    FREQ    FORCE    STIFF
     4    25 0     1 50       74
 DEFL1   DEFL2    DEFL3    DEFL4
  2 01    1 36       74       36
COMMENT STA 1031+75 NBL

      COMPLIANCE----MILS/KIP
TEST# 0     1     2     3     4     5
      +-----+-----+-----+-----+-----+
 1                +
 2                 +
 3                 +
 4                 +
*****       END OF DATA       *****
```

```
TEST DOCUMENTATION  FILE UM05
OPERATOR        SWC
DATE            5 6/91
TIME            1 15
TEMPERATURE   A75P98
TEST LOCATION
UMO TEST SECTION SOIL-CEMENT STA
BILIZED SUBBASE

 TEST#   FREQ    FORCE    STIFF
     1   25 0     1 51     1 27
 DEFL1  DEFL2    DEFL3    DEFL4
  1 13     85       53       31
COMMENT STA 1029+25 SBL

 TEST#   FREQ    FORCE    STIFF
     2   25 0     1 50     1 29
 DEFL1  DEFL2    DEFL3    DEFL4
  1 16     97       56       35
COMMENT STA 1028+25 SBL

 TEST#   FREQ    FORCE    STIFF
     3   25 0     1 52       91
 DEFL1  DEFL2    DEFL3    DEFL4
  1 66   1 23       36       43
COMMENT STA 1028+75 NBL

 TEST#   FREQ    FORCE    STIFF
     4   25 0     1 50     1 00
 DEFL1  DEFL2    DEFL3    DEFL4
  1.49   1 14       85       51
COMMENT STA 1029+75 NBL

   COMPLIANCE----MILS/KIP
TEST# 0    1    2    3    4    5
      +----+----+----+----+----+
 1          +
 2          +
 3            +
 4            +
*****      END OF DATA      *****
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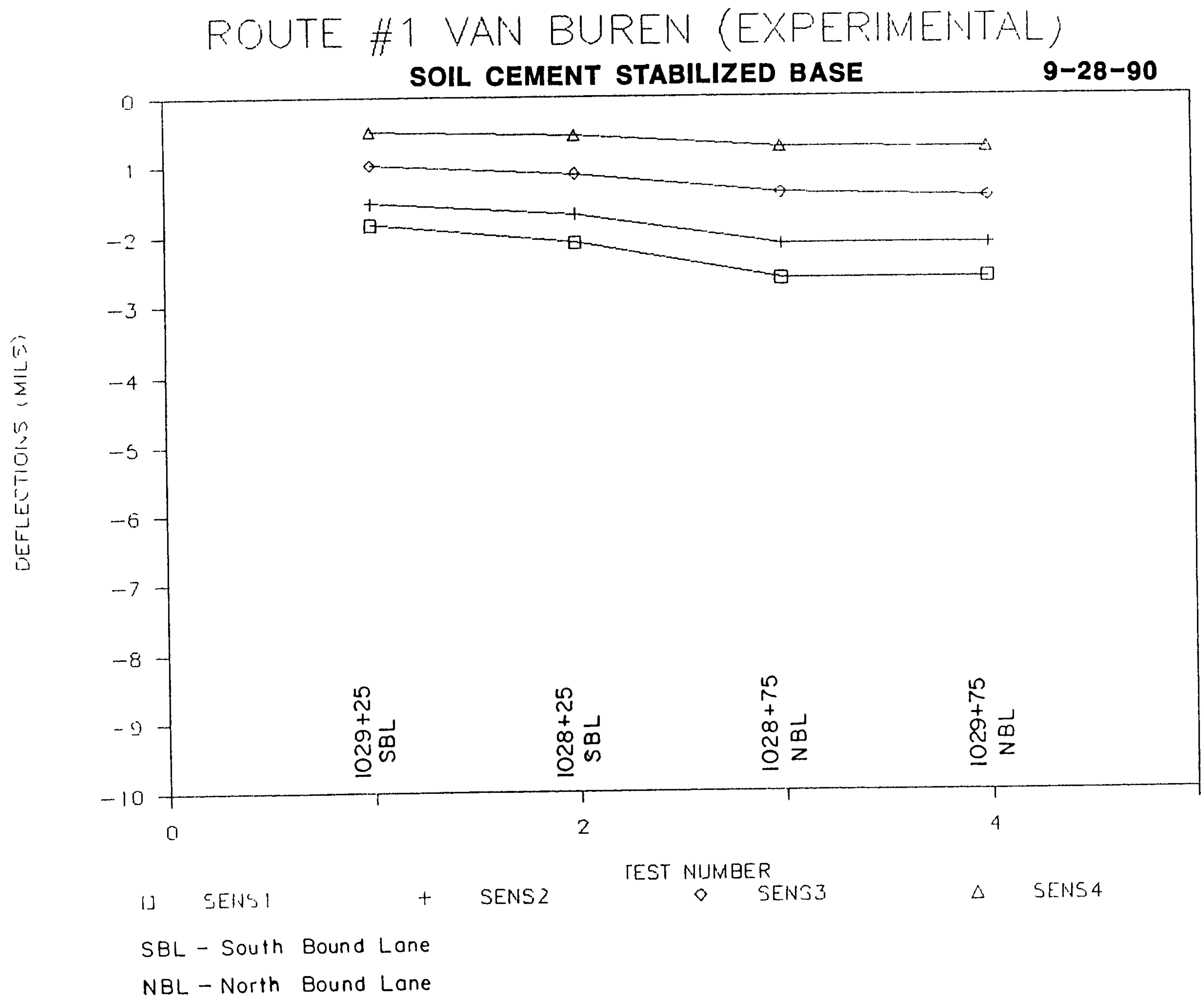
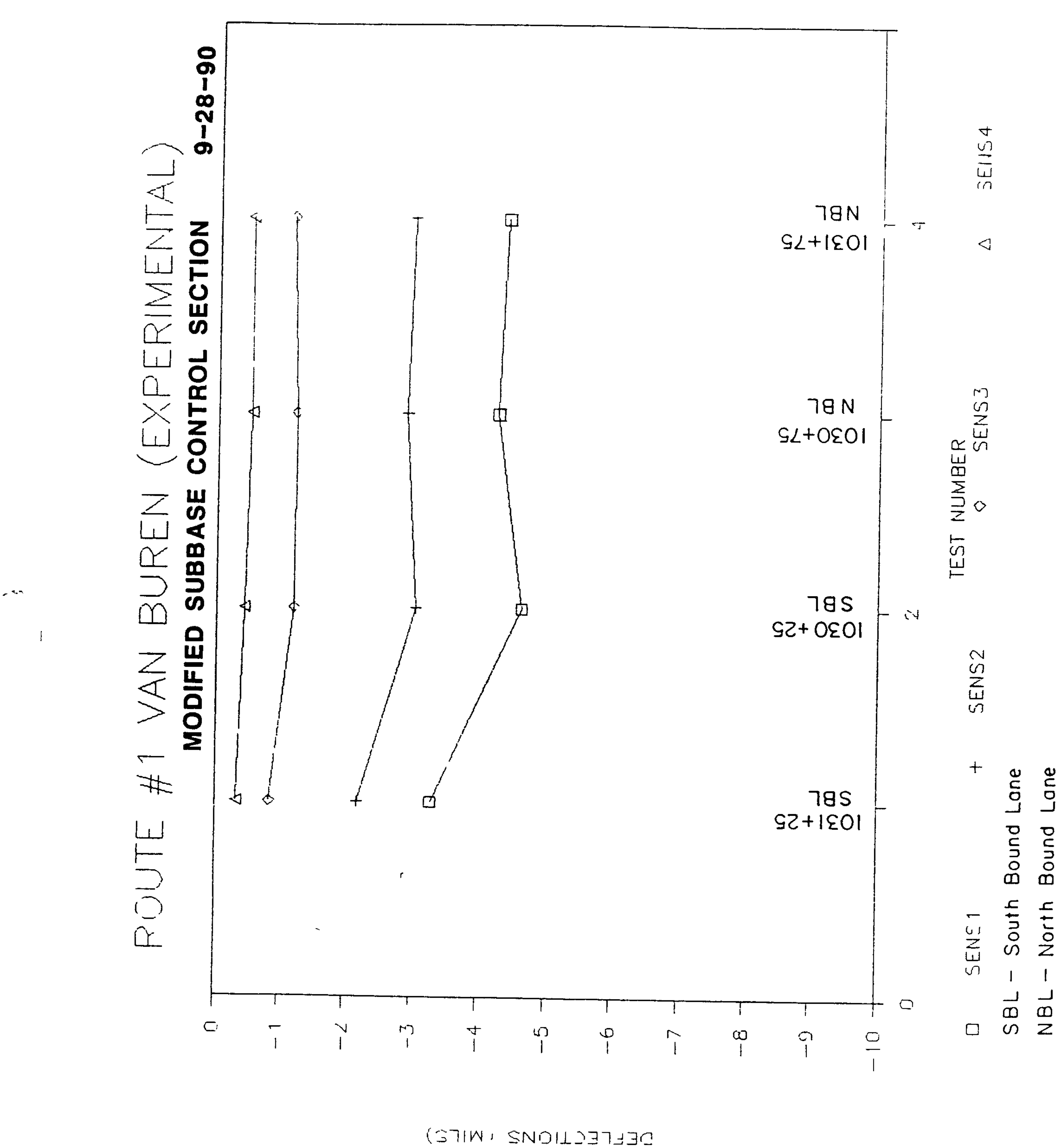
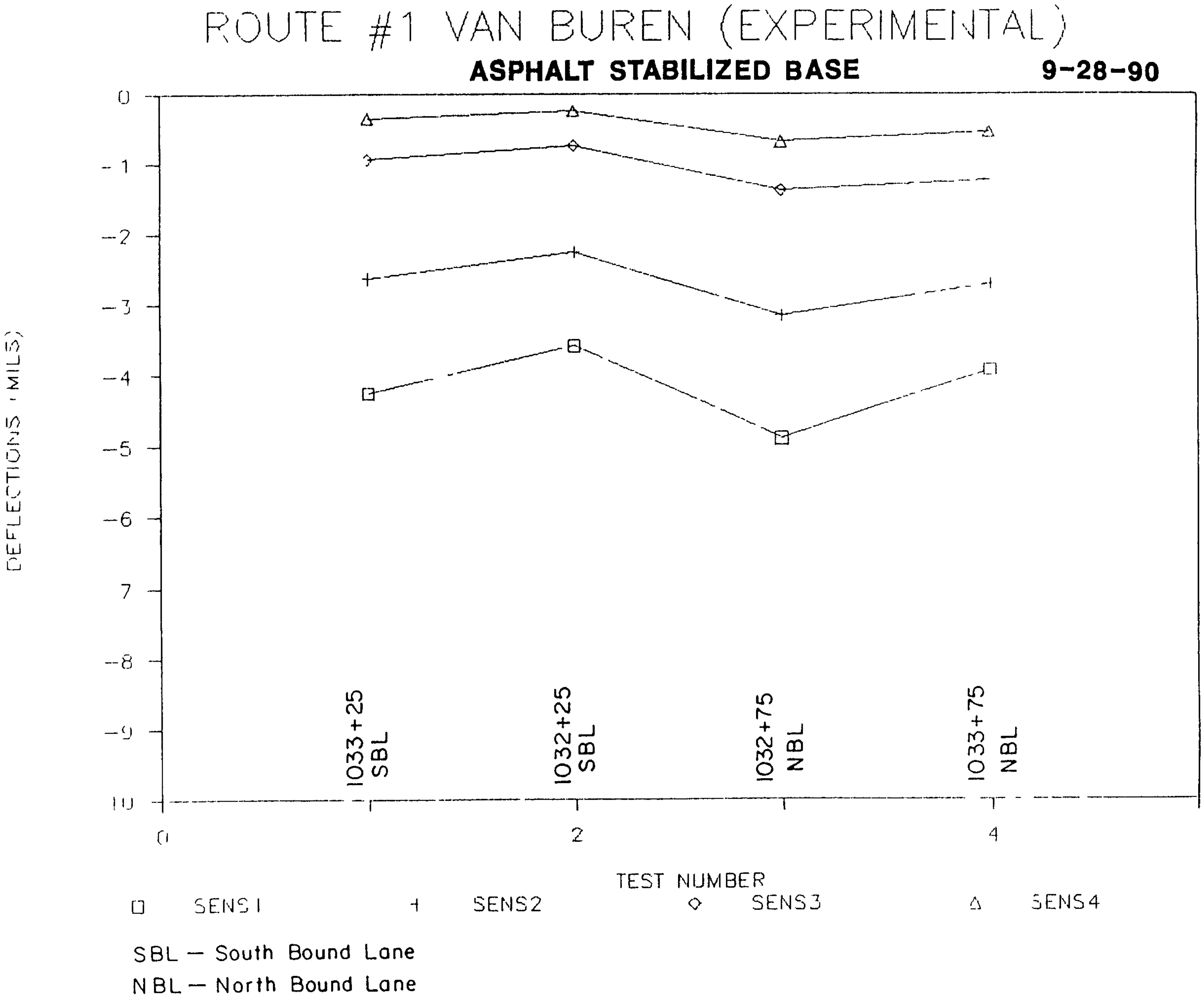

ROUTE #1 VAN BUREN (EXPERIMENTAL)
SOIL CEMENT STABILIZED BASE
9-28-90
DEFLECTIONS (MILS)
0
1
-2
-3
-4
-5
-6
-7
-8
-9
-10
1029+25 SBL
1028+25 SBL
1028+75 NBL
1029+75 NBL
0
2
4
TEST NUMBER
SENS1
SENS2
SENS3
SENS4
SBL - South Bound Lane
NBL - North Bound Lane

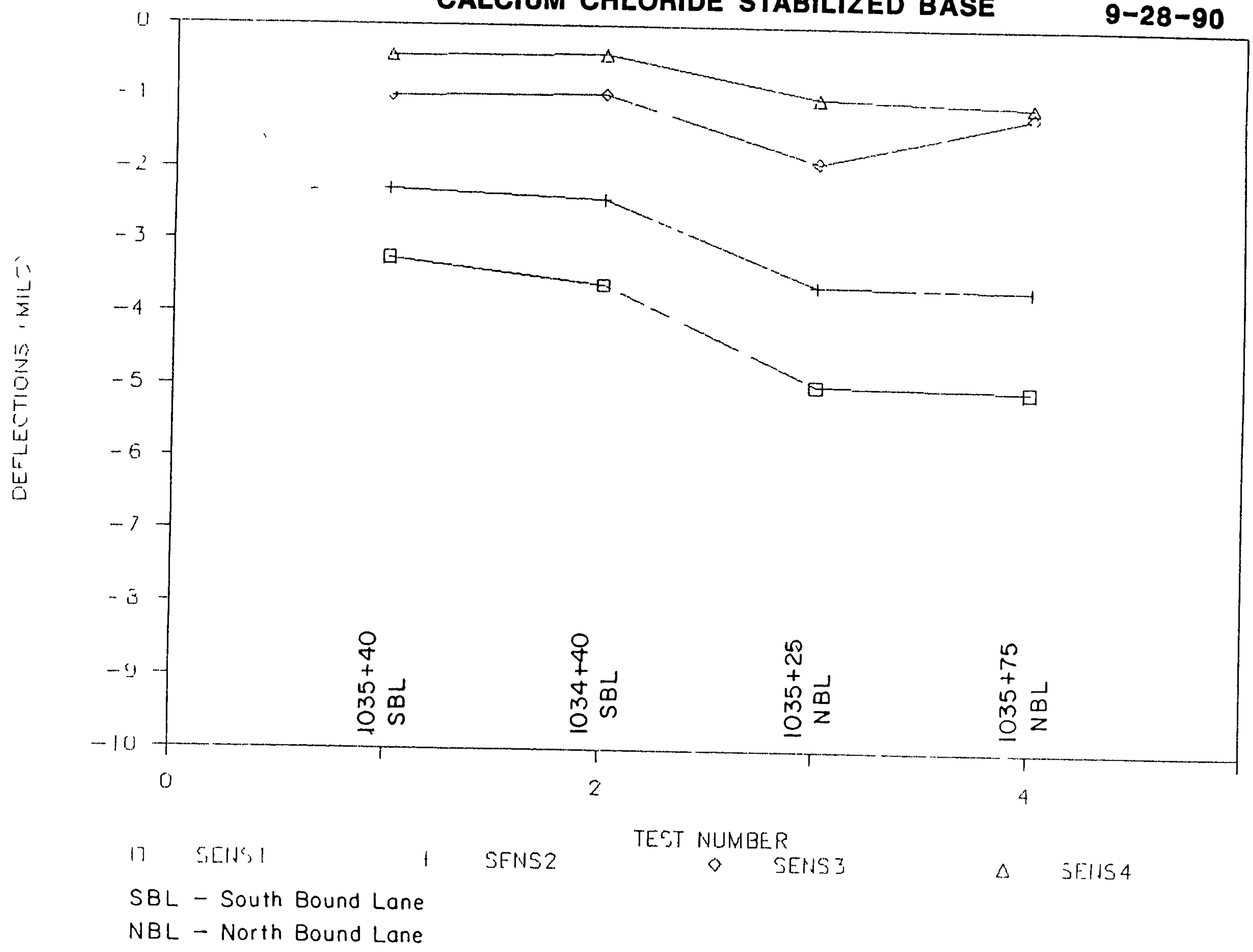

ROUTE #1 VAN BUREN (EXPERIMENTAL)
MODIFIED SUBBASE CONTROL SECTION
9-28-90
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
1031+25 SBL
1030+25 SBL
1030+75 NBL
1031+75 NBL
0
2
4
TEST NUMBER
□ SENS1
+ SENS2
◇ SENS3
△ SENS4
SBL - South Bound Lane
NBL - North Bound Lane


ROUTE #1 VAN BUREN (EXPERIMENTAL)
ASPHALT STABILIZED BASE
9-28-90
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
7
-8
-9
10
1033+25 SBL
1032+25 SBL
1032+75 NBL
1033+75 NBL
0
2
4
TEST NUMBER
SENS1
SENS2
SENS3
SENS4
SBL — South Bound Lane
NBL — North Bound Lane

ROUTE #1 VAN BUREN (EXPERIMENTAL)

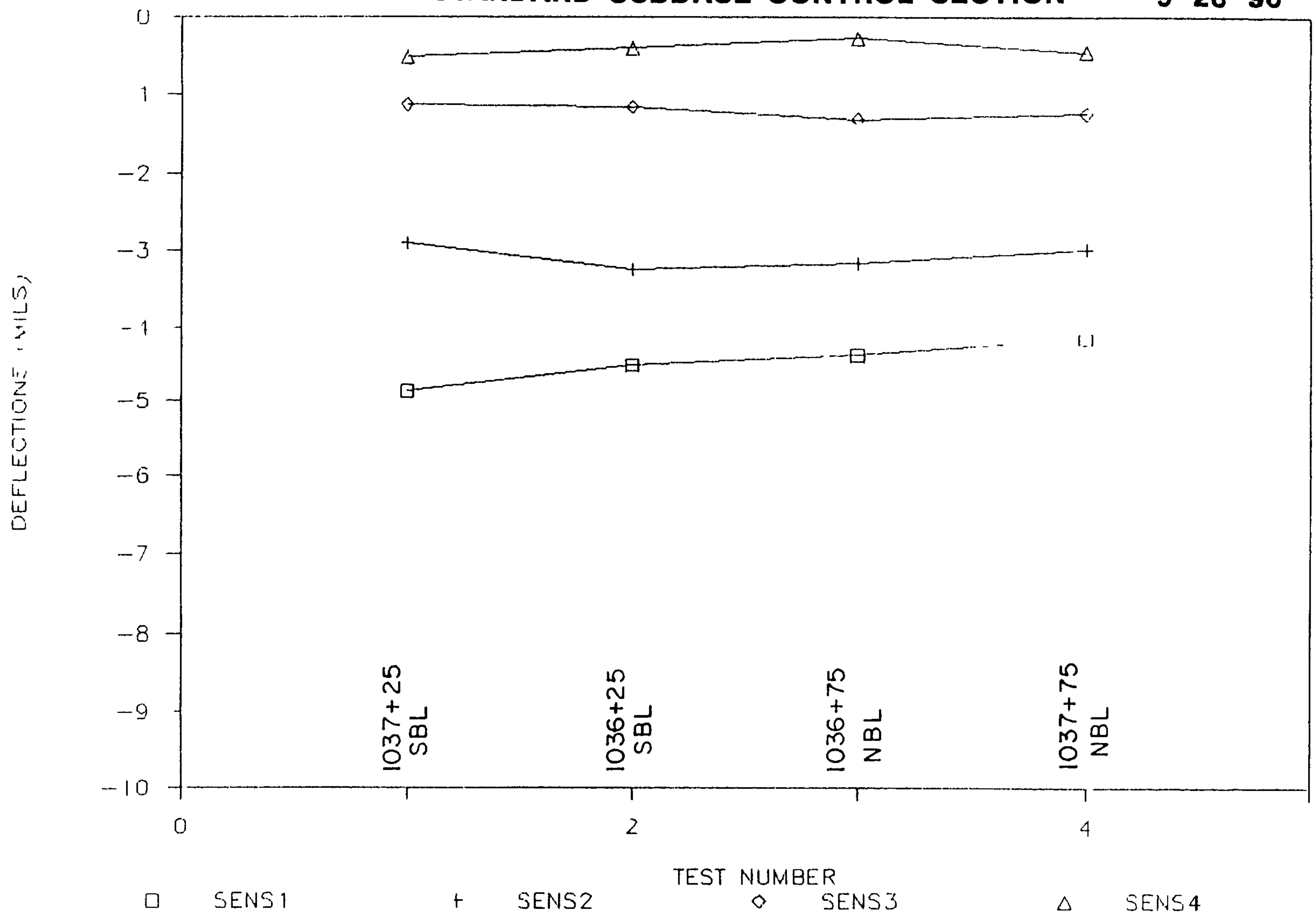
CALCIUM CHLORIDE STABILIZED BASE

9-28-90

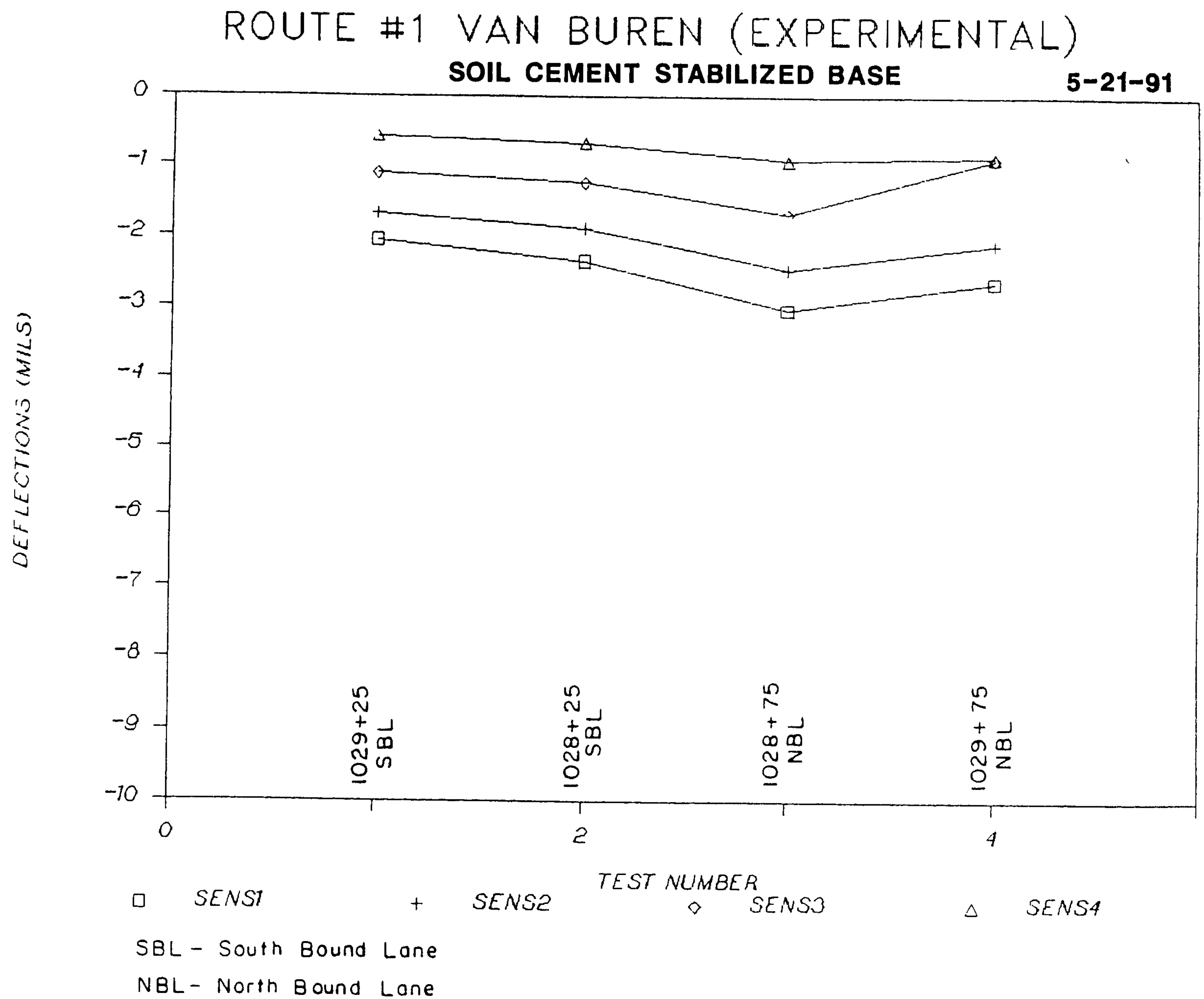


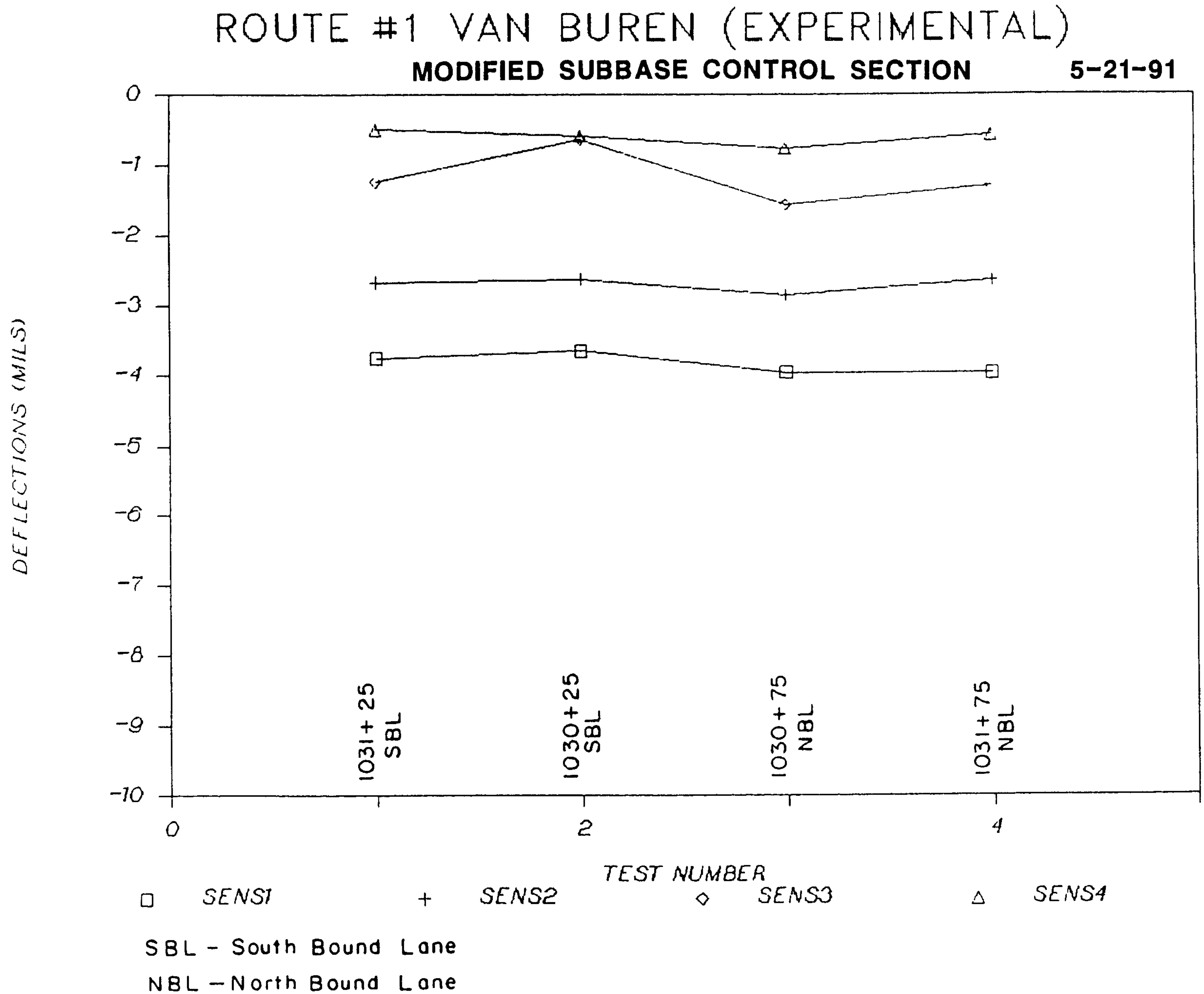
ROUTE #1 VAN BUREN (EXPERIMENTAL)

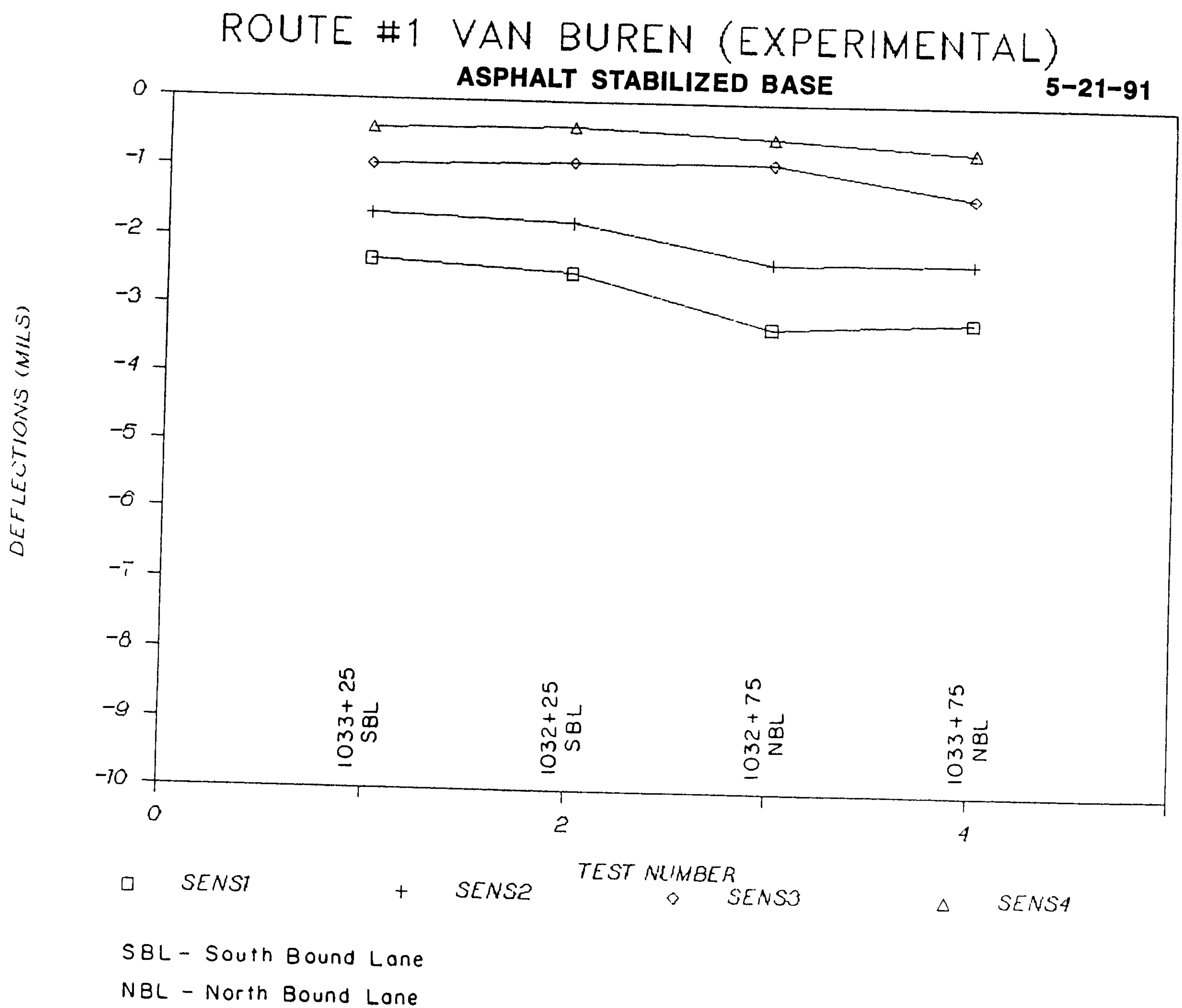
STANDARD SUBBASE CONTROL SECTION **9-28-90**



SBL - South Bound Lane
NBL - North Bound Lane

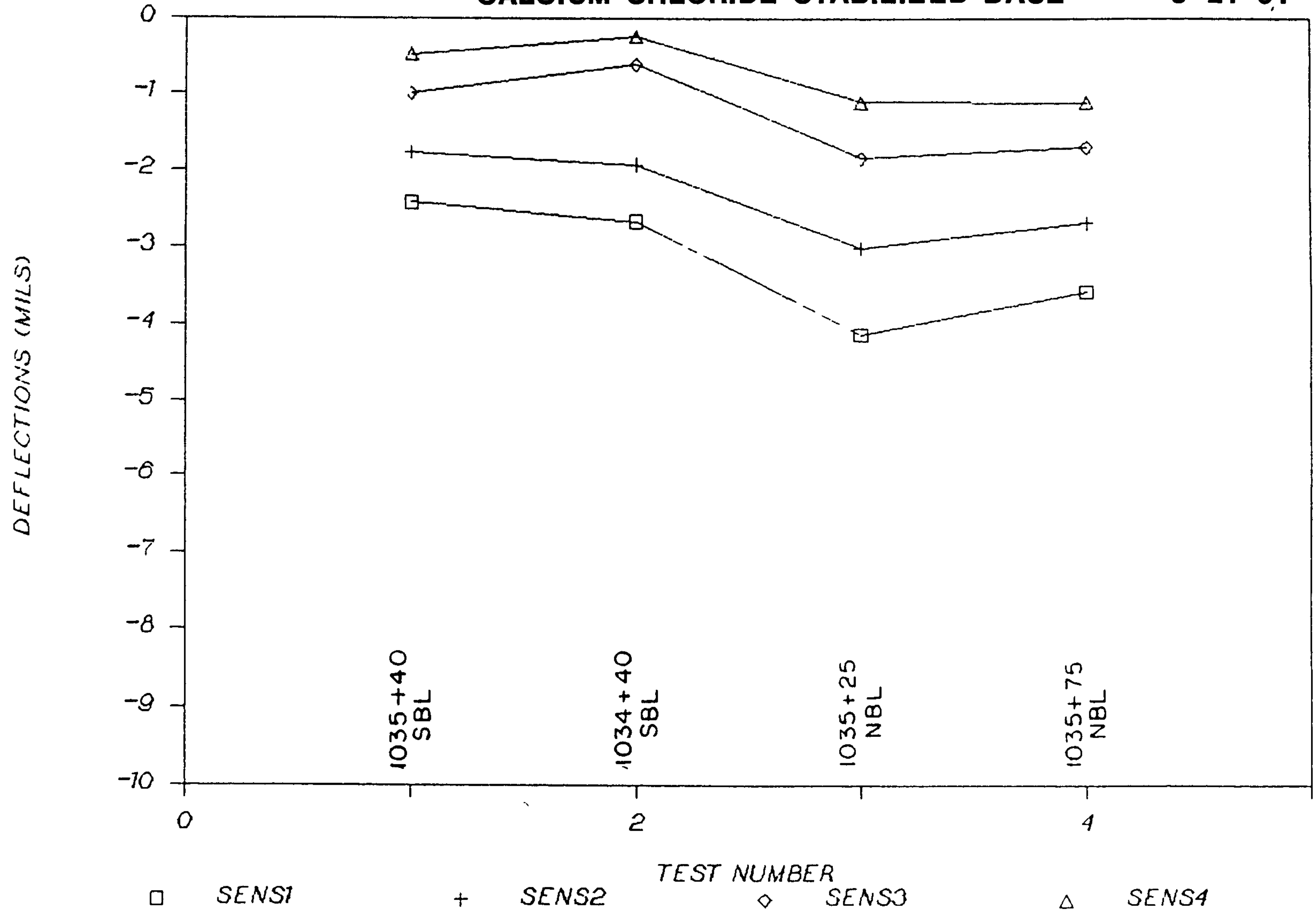

ROUTE #1 VAN BUREN (EXPERIMENTAL)
SOIL CEMENT STABILIZED BASE
5-21-91
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
1029+25 SBL
1028+25 SBL
1028+75 NBL
1029+75 NBL
0
2
4
TEST NUMBER
SENS1
SENS2
SENS3
SENS4
SBL - South Bound Lane
NBL - North Bound Lane


ROUTE #1 VAN BUREN (EXPERIMENTAL)
MODIFIED SUBBASE CONTROL SECTION
5-21-91
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
1031+ 25 SBL
1030+25 SBL
1030+ 75 NBL
1031+ 75 NBL
0
2
4
TEST NUMBER
SENS1
SENS2
SENS3
SENS4
SBL - South Bound Lane
NBL - North Bound Lane


ROUTE #1 VAN BUREN (EXPERIMENTAL)
ASPHALT STABILIZED BASE
5-21-91
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
1033+25 SBL
1032+25 SBL
1032+75 NBL
1033+75 NBL
0
2
4
TEST NUMBER
SENS1
SENS2
SENS3
SENS4
SBL - South Bound Lane
NBL - North Bound Lane

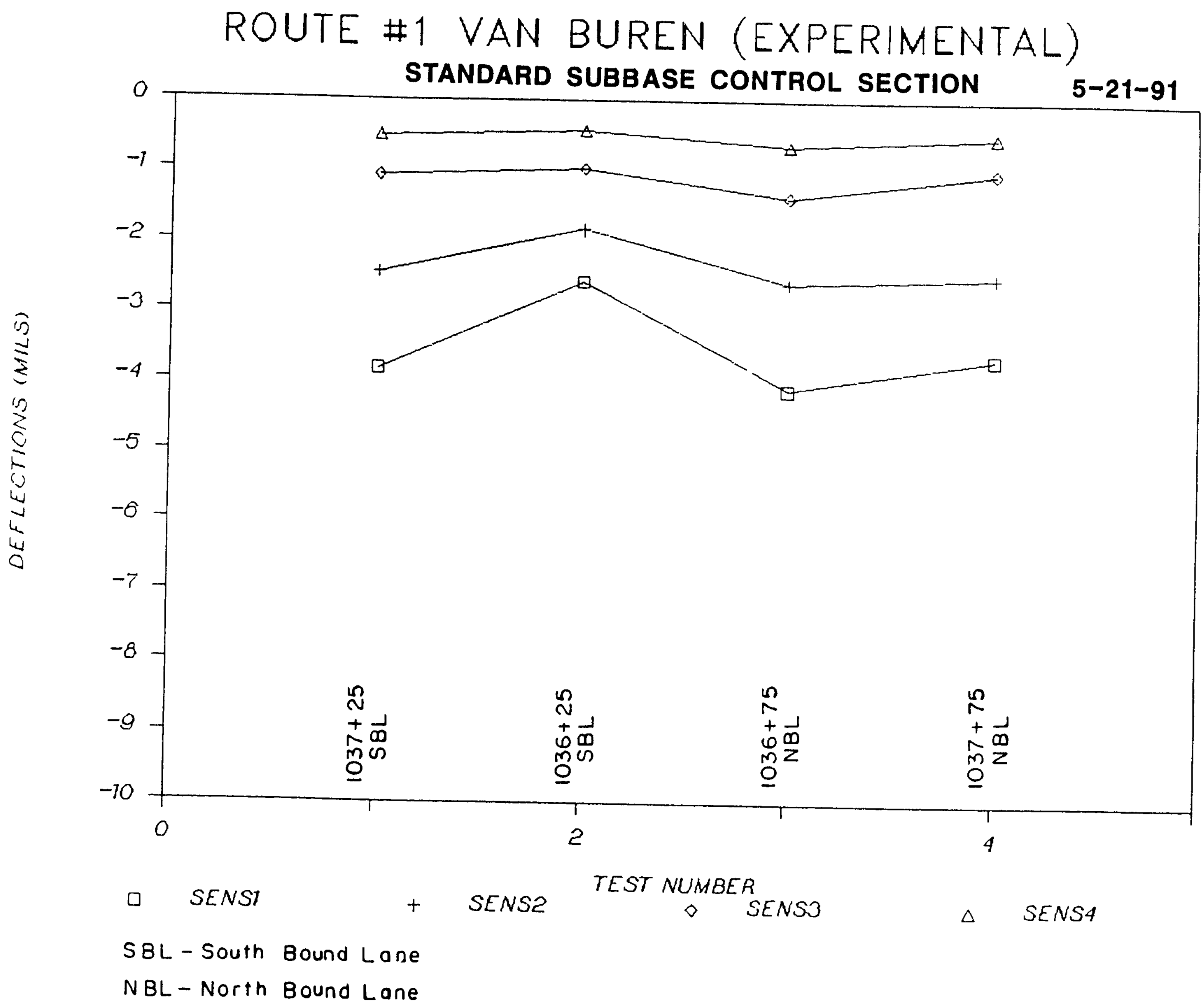
ROUTE #1 VAN BUREN (EXPERIMENTAL)

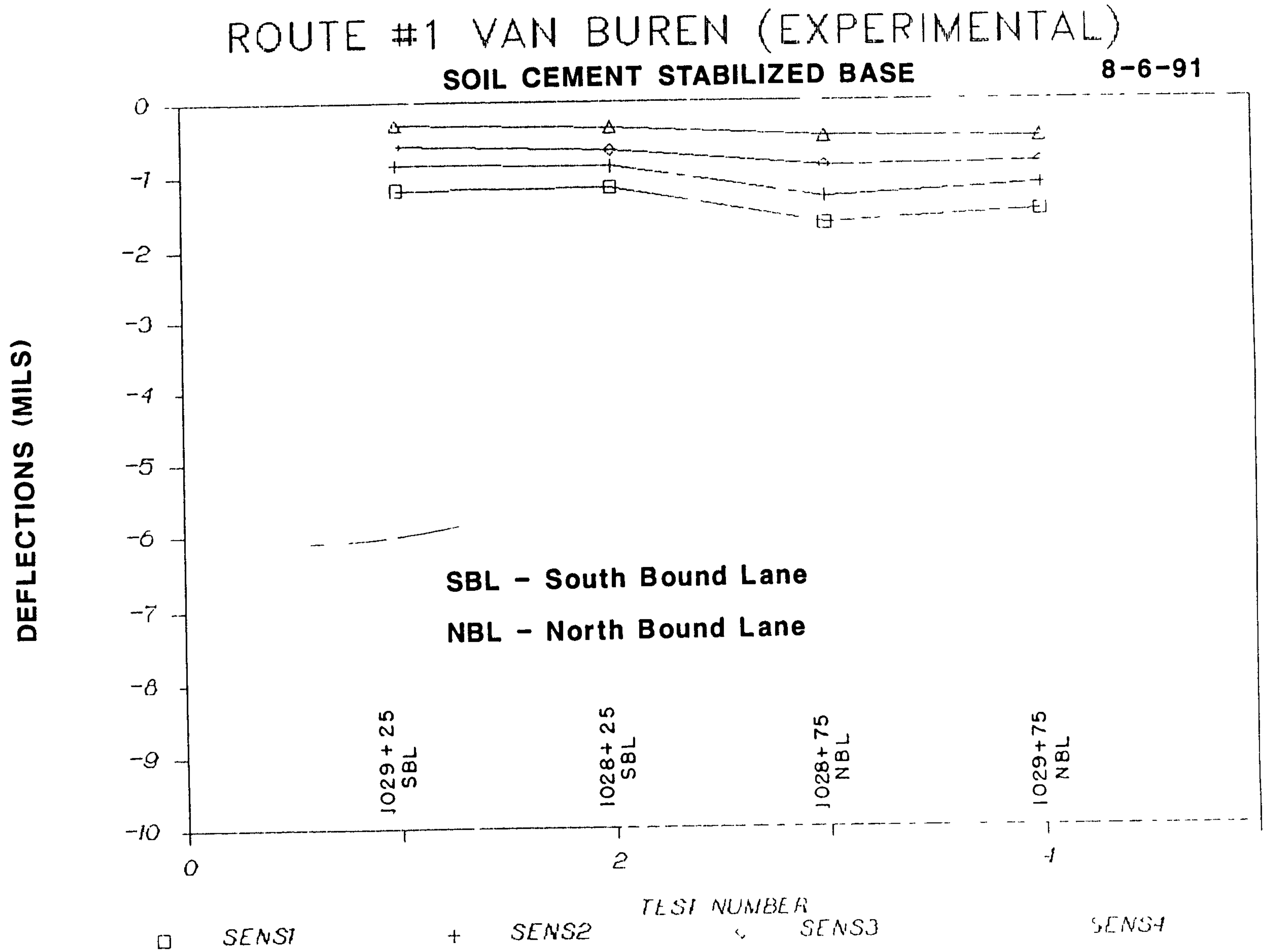
CALCIUM CHLORIDE STABILIZED BASE **5-21-91**

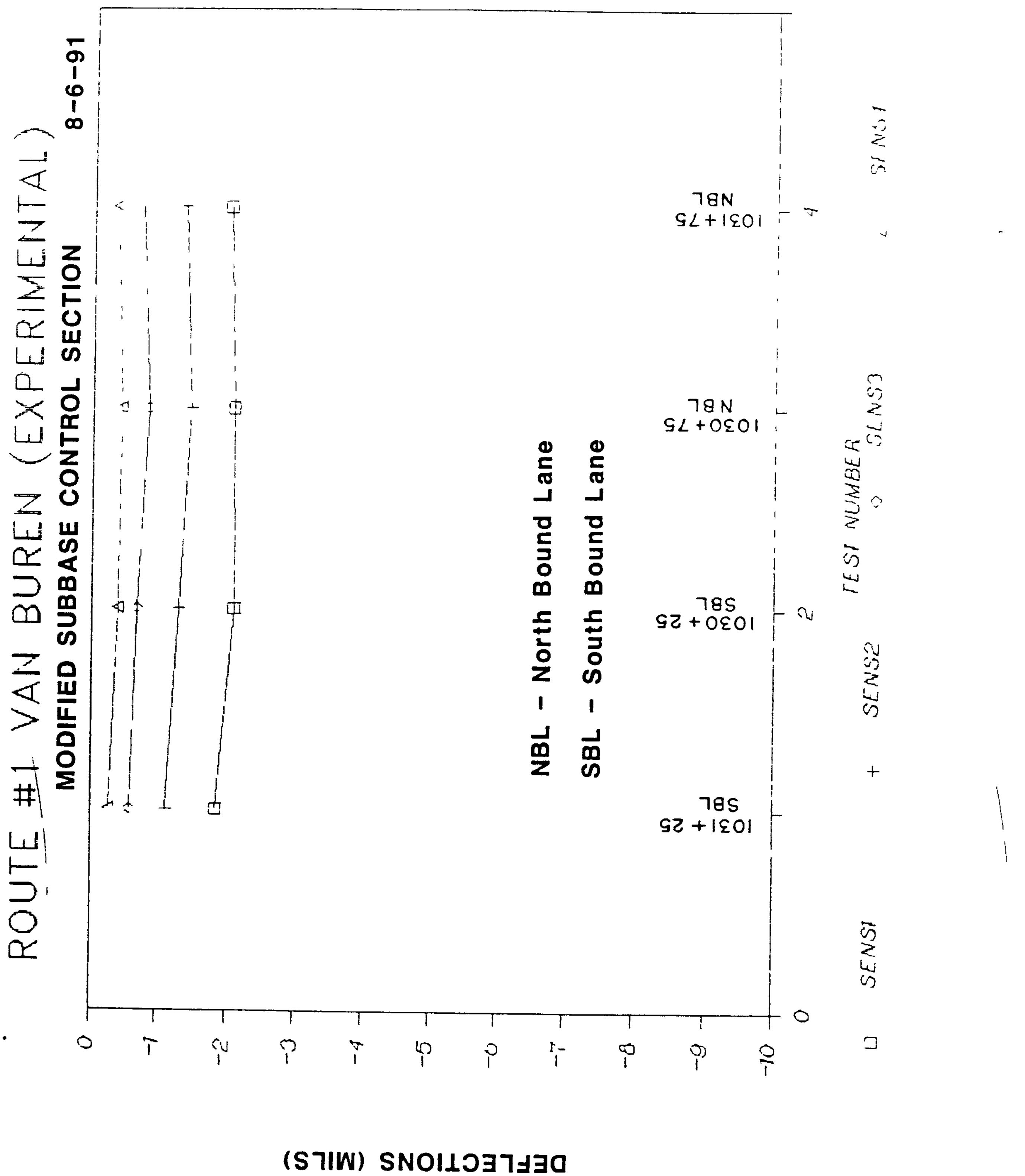


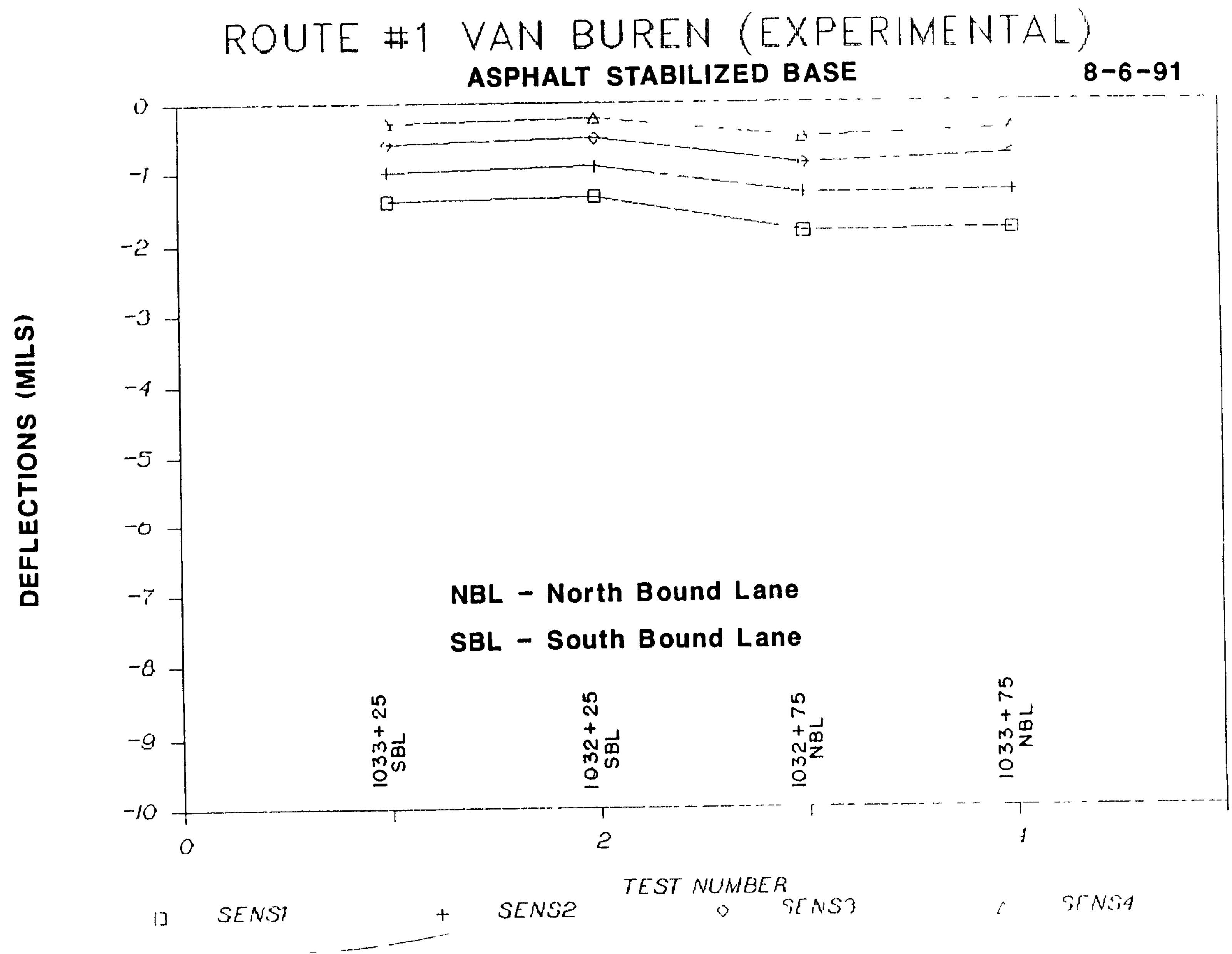
SBL – South Bound Lane

NBL – North Bound Lane

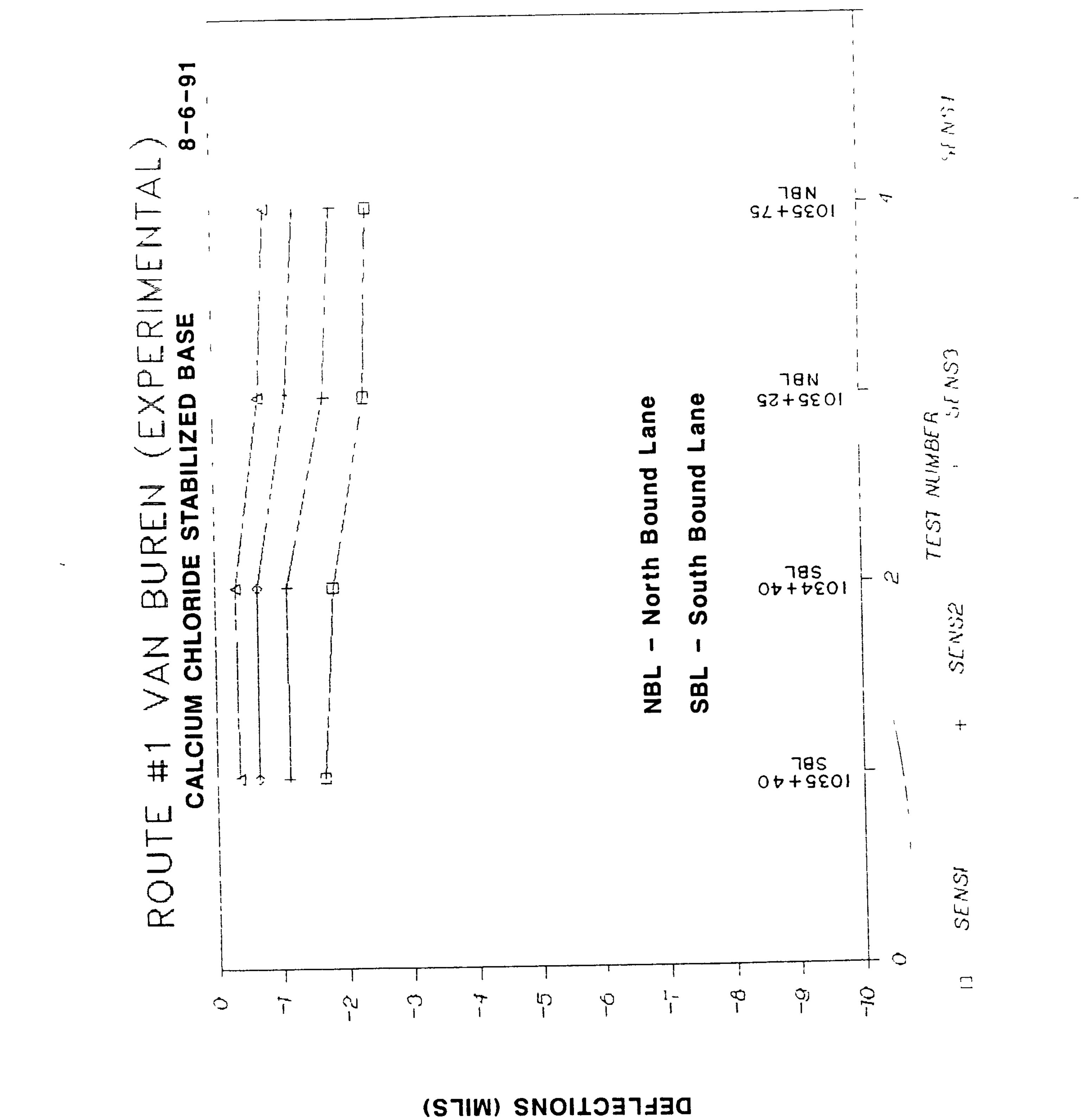

ROUTE #1 VAN BUREN (EXPERIMENTAL)
STANDARD SUBBASE CONTROL SECTION
5-21-91
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
1037+25 SBL
1036+25 SBL
1036+75 NBL
1037+75 NBL
0
2
4
TEST NUMBER
SENS1
SENS2
SENS3
SENS4
SBL - South Bound Lane
NBL - North Bound Lane

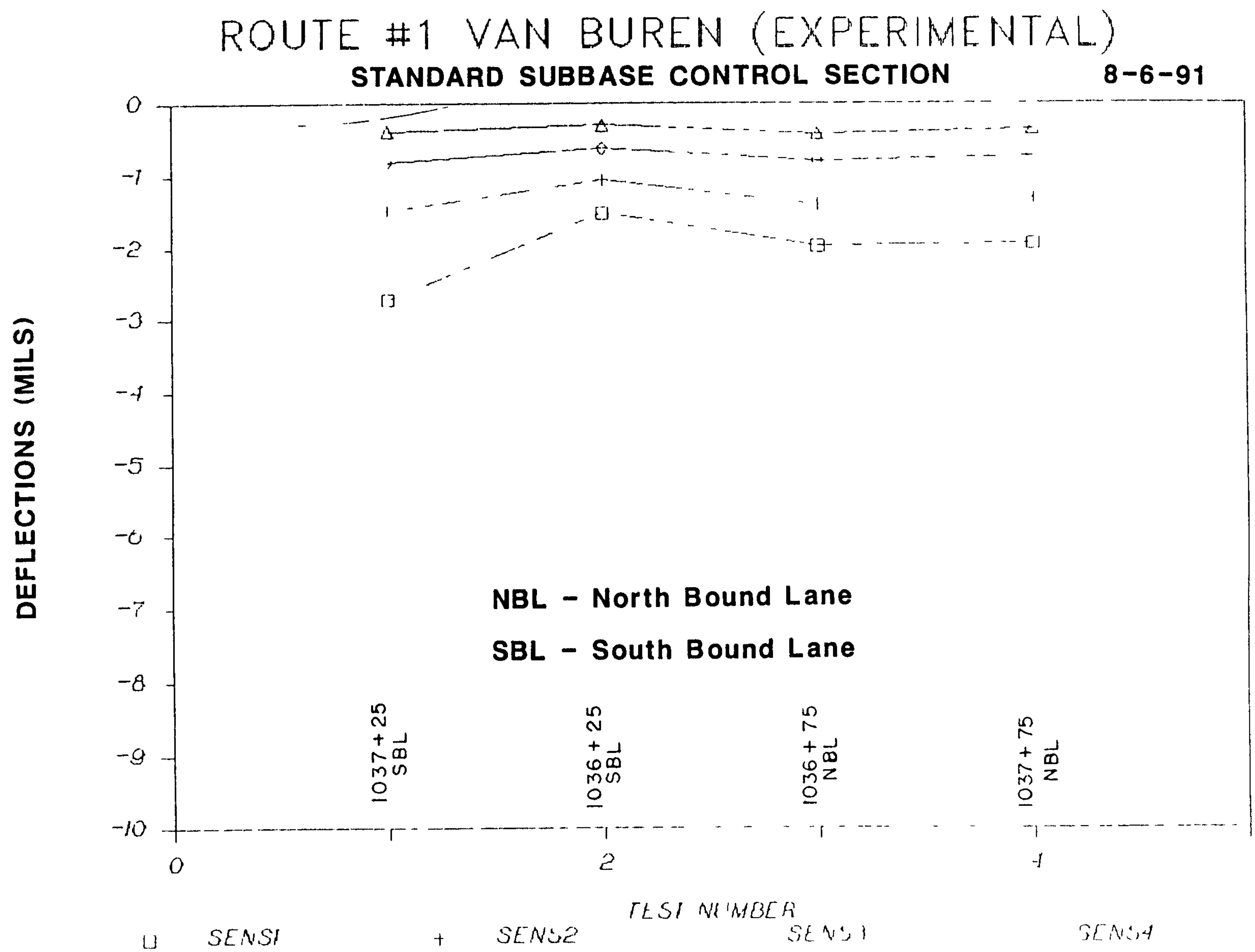

ROUTE #1 VAN BUREN (EXPERIMENTAL)
SOIL CEMENT STABILIZED BASE
8-6-91
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
SBL - South Bound Lane
NBL - North Bound Lane
1029 + 25 SBL
1028+ 25 SBL
1028+ 75 NBL
1029+75 NBL
0
2
TEST NUMBER
SENS1
SENS2
SENS3
SENS4


ROUTE #1 VAN BUREN (EXPERIMENTAL)
MODIFIED SUBBASE CONTROL SECTION
8-6-91
NBL - North Bound Lane
SBL - South Bound Lane
1031+25 SBL
1030+25 SBL
1030+75 NBL
1031+75 NBL
TEST NUMBER
DEFLECTIONS (MILS)
SENS1
SENS2
SENS3
SENS4
0
2
4
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10



ROUTE #1 VAN BUREN (EXPERIMENTAL)
ASPHALT STABILIZED BASE
8-6-91
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
NBL - North Bound Lane
SBL - South Bound Lane
1033+25 SBL
1032+25 SBL
1032+75 NBL
1033+75 NBL
0
2
1
TEST NUMBER
SENS1
SENS2
SENS3
SENS4


ROUTE #1 VAN BUREN (EXPERIMENTAL)
CALCIUM CHLORIDE STABILIZED BASE
8-6-91
NBL - North Bound Lane
SBL - South Bound Lane
1035+40 SBL
1034+40 SBL
1035+25 NBL
1035+75 NBL
TEST NUMBER
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
1
2
3
4
SENS1
SENS2
SENS3
SENS4


ROUTE #1 VAN BUREN (EXPERIMENTAL)
STANDARD SUBBASE CONTROL SECTION
8-6-91
DEFLECTIONS (MILS)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
NBL - North Bound Lane
SBL - South Bound Lane
1037+25 SBL
1036+25 SBL
1036+75 NBL
1037+75 NBL
0
2
4
TEST NUMBER
SENS1
SENS2
SENS3
SENS4

APPENDIX F

CALCIUM CHLORIDE CONTENT CALCULATIONS

UNIVERSITY OF MAINE

Department of Plant and Soil Sciences

Deering Hall
Orono Maine 04469 0118

From: Michael Moreau
103 Boardman Hall
Campus

Job # 2318
Date Rec : 9/26/90
Date Printed· 10/11/90

Sample Type Gravel

Sample	Ca (mg/kg)	Cl (mg/kg)
#1	1880	5780
#2	1840	4980
#3	1870	7670
#4	1780	4000
#5	1990	11300
#6	1840	4980
#7	350	6.8
#8	310	9.5
#9	380	23
#10	2010	9330
#11	1890	6930
#12	1930	6930
#13	1980	8230

All results on dry-weight basis. Ca quantitated on I C.P. spectrometer and Cl quantitated by chloride ion-selective electrode with a doulbe-junction reference electrode.

William P Cook

William P Cook
Assistant Chemist

$CaCl_2$ CONTENT CALCS

- Based on 10/11/90 Results from Dept. of Plant & Soil Sciences (previous page)
  * Avg. Ca (treated soil) = 1901 mg/kg
  * Background Ca (untreated soil) = 347 mg/kg

→ Avg. Added Ca (treated soil) = 1901 − 347 = 1554 mg/kg

- Field Appl. Rate: .75 gal/yd²

$.75\ gal/yd^2 \div [9 \times 0.5] = 0.167\ gal/ft^3$

$0.167\ gal/ft^3 \times 8.3453\ \#/gal = \dfrac{1.39\ \#\ H_2O/ft^3}{135\ \#\ soil/ft^3} \times 100\% = 1.0\%$

$0.167\ \frac{gal}{ft^3} \div 135\ \#/ft^3 = 0.00124\ gal/lb.$

$0.00124\ gal/lb \times 3.9\ \#\ CaCl_2/gal = 0.00482\ \#\ CaCl_2/lb$

$\dfrac{1.554\ g\ Ca}{kg} \times 2.77 = 4.30\ g\ CaCl_2/kg$

2.77 → total Molecular wt Proportion

$3.9\ \#\ CaCl_2/gal \times 454\ g/lb = 1771\ g/gal$

EQN. 1 $\dfrac{4.30\ g/kg}{1771\ g/gal} = 0.00243\ gal/kg$

$0.00243\ gal/kg \times 3785.434\ ml/gal \div 2.205\ lb/kg$

$= 4.17\ ml/lb$ dry soil

82

UNIVERSITY OF MAINE

Department of Plant and Soil Sciences

Deering Hall
Orono, Maine 04469 0118

From: Michael Moreau
103 Boardman Hall
Campus

Job # 2318
Date Rec 9/26/90
Date Printed. 10/22/90

Sample Type: Gravel

Sample	Ca (mg/kg)	Cl (mg/kg)	Na (mg/kg)
#1	1600	2900	48
#2	1400	2500	39
#3	1600	2500	42
#4	1200	2000	34
#5	2800	5700	91
#6	1500	2500	43
#7	180	6.8	6.5
#8	150	9.5	7.3
#9	190	23	9.3
#10	2100	4700	69
#11	1700	3500	58
#12	1800	3500	56
#13	2000	4100	56

All results on dry-weight basis. Ca quantitated on I.C.P. spectrometer and Cl quantitated by chloride ion-selective electrode with a doulbe-junction reference electrode. Na determined on atomic-absorption spectrophotometer

William P Cook

William P Cook
Assistant Chemist

$CaCl_2$ CONTENT CALCS

– Based on 10/22/98 Results from Dept. of Plant & Soil Sciences (previous page)

1. Subtract Avg Background Ca from Each treated sample → yield added Ca
2. Subtract Avg Background Cl from Each treated sample → yield added Cl
3. Combine Calculated Ca & Cl for each treated sample – Average all samples

→ Average added $CaCl_2$ = 4974 mg/kg

Substitute into EQN 1 – previous calcs

$$\frac{4.97\ g/kg}{1771\ g/gal} = 0.00281\ gal/kg$$

0.00281 gal/kg × 3785.434 ml/gal ÷ 2.205 lb/kg

= 4.82 ml/lb dry soil

APPENDIX G

LONG TERM MONITORING PROGRAM

Long Term Monitoring Program

Long term monitoring is critical to evaluating the effectiveness of the soil cement, asphalt, and calcium chloride test sections For comparison the modified subbase aggregate and standard subbase aggregate control sections should also be monitored. Monitoring should continue for a minimum of 15 years after construction. At that time, the desirability of continuing the monitoring program should be evaluated. The location of each test and control section is given in Table G-1.

Table G-1
Location of test and control sections.

Section	Stations
Soil Cement	1028+00 to 1030+00
Modified Subbase Control	1030+00 to 1032+00
Asphalt	1032+00 to 1034+00
Calcium Chloride	1034+20 to 1036+20
Standard Subbase Control	1036+20 to 1038+20

In addition to the test and control sections listed above there is an untreated zone from Stations 1034+00 to 1034+20. No monitoring will be performed in this 20-ft long zone

It is recommended that the monitoring program include an annual evaluation with the ARAN vehicle and the road rater, and a biannual elevation and crack survey. In addition, if rutting in excess of 1 to 2 inches should develop, excavations should be made to determine the cause of the rutting. For the calcium chloride stabilized section, there is some possibility that the calcium chloride would be washed away with time. This should also be monitored. Each component of the monitoring program is discussed below along with the recommended procedure to evaluate the data.

The entire length of the experimental test sections should be evaluated on an annual basis with the ARAN vehicle. The ARAN vehicle stationing should be reset at the beginning of the experimental test section to ensure that the stationing coincides with the location of the individual test sections. Signs at Stations 1028+00 and 1038+20 indicate the beginning and end of the test sections, respectively. The ARAN data should be used to evaluate roughness, rutting, and overall performance. The international roughness index should be measured over 50-ft long intervals in both the north and south bound lanes, giving eight readings for each test section

The average roughness for each section should be computed and plotted versus years in service. A profile of the roadway surface should be plotted at the stations given in Table G-2 A format similar to that shown in Appendix C should be used. The maximum rut depth should be computed in both the north and south bound lanes of each section, giving eight rut depths for each test section These eight rut depths should be averaged and plotted versus years in service. The overall pavement condition in each 200-ft long section should be evaluated using standard MDOT procedures. The pavement condition rating should be plotted versus years in service for each test and control section.

Table G-2
Stations for profiles and elevation surveys

Station	Section
1028+50 1029+50	Soil Cement
1030+50 1031+50	Modified Subbase Control
1032+50 1033+50	Asphalt
1034+50 1035+50	Calcium Chloride
1036+50 1037+50	Standard Subbase Control

The sections should be evaluated on an annual basis with the road rater at the stations and lanes indicated in Table G-3. There are four stations in each section. Note that the stationing in the calcium chloride section is slightly different than for the other sections. The sensors should be positioned in the outer wheel path. The measurements should be taken in July or August. The deflection measured by sensor 1 is felt to be most representative of the performance of the stabilized base The average of the four sensor 1 readings taken in each test section should be plotted versus years in service as shown on Fig. G-1. The initial readings on the completed wearing surface were taken on August 6, 1991 These readings are shown on Fig. G-1.

Surveyed elevation profiles should be made in the summer of each odd numbered year at the stations indicated in Table G-2. At each station elevation measurements should be taken at the center line and at 2-ft intervals away from the center line across the full width of the north and south bound travel lanes. The sections should be plotted at a reasonable scale, such as a vertical scale of 1 in. equals 0.1 ft and a horizontal scale of

Table G-3
Stations for road rater measurements.

Station and Lane	Section
1028+25 SBL 1028+75 NBL 1029+25 SBL 1029+75 NBL	Soil Cement
1030+25 SBL 1030+75 NBL 1031+25 SBL 1031+75 NBL	Modified Subbase Control
1032+25 SBL 1032+75 NBL 1033+25 SBL 1033+75 NBL	Asphalt
1034+40 SBL 1035+25 NBL 1035+40 SBL 1035+75 NBL	Calcium Chloride
1036+25 SBL 1036+75 NBL 1037+25 SBL 1037+75 NBL	Standard Subbase Control

Note: SBL = south bound lane
NBL = north bound lane

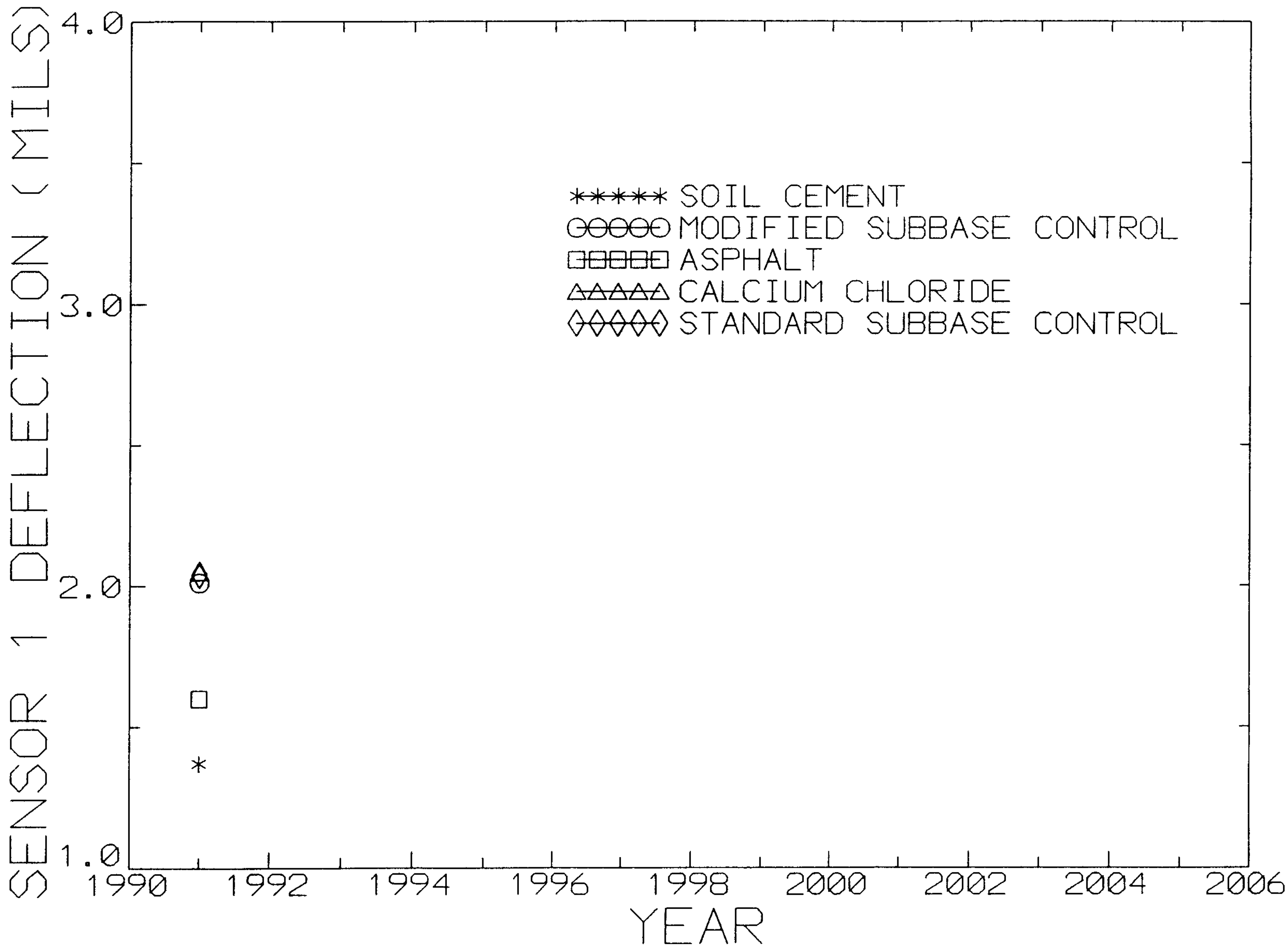


Fig G-1 Average road rater sensor 1 deflection

1 in. equals 0 4 ft. Readings from successive years should be plotted on the same cross section so that any deformation of the pavement surface with time can be monitored. The survey data should also be used to compute the maximum rut depth in the north and south bound lane of each station Rut depth should be computed using the same procedure used by the ARAN vehicle This will result in four rut depths for each section. The values should be averaged and plotted versus years in service

Any cracking which develops in the travel lanes should be mapped in each odd numbered year. The cracking should be plotted on a plan view of each test section. Results from crack surveys in succeeding years should be shown on the same plan view with a different color being use for cracks which develop in each survey year. The total lineal feet of cracking in the travel lanes of each section should be plotted versus years in service.

Finally, if significant rutting develops, the cause of the rutting should be investigated. To facilitate this evaluation, elevation measurements were taken during construction at the top of subgrade, mid-depth of subbase, top of completed subbase or stabilized course just prior to paving, top of binder course, and top of completed wearing surface at the stations given in Table G-2. Results of the surveys are given in Appendix C. The elevation survey at the mid-depth of the subbase course was marked by placing a layer of 3/4-in. minus crushed stone on top of the surveyed section. The location of the other sections will be evident since they are at the interface between two dissimilar materials. To investigate the source of rutting, test trenches should be excavated across the full width of the travel lanes at the stations in Table G-2 which had experienced significant rutting. The previously surveyed sections will be evident in the sidewalls of the trench An elevation profile with readings taken at the centerline and 2-ft intervals away from the centerline should be made for each section. Readings should be taken on both sides of the trench and averaged. These profiles should be compared to the original profiles to reveal the course in which the rutting is occurring. In addition, the visual condition of the stabilized course should be examined.

For the calcium chloride section, there is some concern that the calcium chloride will be washed away with time. This should be investigated after 5, 10 , and 15 years in service by drilling 16 shallow test holes in the travel lanes of this section. Samples should taken of the calcium chloride stabilized base course and the underlying subbase course. These samples should be analyzed for total calcium and total chloride content The calcium and chloride contents of the samples from the calcium chloride stabilized base should be compared to the levels measured at the end of construction which are given in Table 6.2 and Appendix F. The calcium and chloride contents of the samples from the underlying subbase course should be compared to the naturally occurring levels which are also given in Appendix F